URBAN WATER SUPPLY AND SANITATION (SECTOR) PROJECT

DESIGN GUIDELINES

PROJECT MANAGEMENT OFFICE

PANIPOKHARI, KATHMANDU

2021

Governmet of Nepal
Ministry of Water Supply
Department of Water supply and Sewerage Management
Panipokhari, Kathmandu

Phone No: 01-4413744
01-4417609
01-4419670
01-4420908
Fax: 01-4419802

Government of Nepal
Ministry of Water Supply
Department of Water Supply & Sewerage Management
Panipokhari, Kathmandu

Ref. No.:

Foreword

Nepal aims to achieve universal access to safe, quality,and functional water, sanitation and hygiene (WASH) services in line with SDG 6 targets. While the country has been able to make incremental progress over the years in water and sanitationservices, many dysfunctional projects impede functional access and, as a result, some of the citizens of Nepal are deprived of safe, sufficient, accessible, acceptable, and affordable water and sanitation services.

Good design is the foundation for effective and sustainable water and sanitation solutions. Systematic planning and robust technical design are essential to ensure functional water and sanitation services at thelowest possible lifecycle cost while also meeting social and environmental safeguards.

The Design Guidelines, which was first prepared by the Second Small Towns Water Supply and Sanitation Sector Project in 2011, was revised and updated in 2015 to promote standardization, quality assurance, and adherence to national standards and good practices. This third revised edition of the Design Guidelines is further updated especially with inclusion of climate resilient designs and Supervisory Control and Data Acquisition (SCADA) in 2021. It is also intended to facilitate a more strategic, integrated planning process, and to embed sector policies in implementation practices.

Aimed at providing practical guidance and knowledge to aid similarand forthcoming projects under Department of Water Supply and Sewerage Management and other WASH agencies across the sector to streamline design processes for urban areas, I believe, this Design Guidelines will be found useful at different stages of the planning and design processes, from developing a strategy for service delivery and investment planning to the feasibility studies and detailed designs.

Continuous learning and instituting improvements are part of people centered development. This Design Guideline can be improved based on the experiences gained under the Urban Water Supply and Sanitation (Sector) Project (UWSSP), and other similar projects.Any suggestion for the improvement of this Design Guidelines from sector institutions and professionals are always welcome.

Tiresh Prasad Khatri
Director General
Department of Water Supply and Sewerage Management (DWSSM)

Government of Nepal
Ministry of Water Supply
Department of Water Supply and Sewerage Management
Urban Water Supply and Sanitation (Sector) Project
PROJECT MANAGEMENT OFFICE
Panipokhari, Kathmandu

Ref No.:

Acknowledgment

The Design Guidelines was first prepared in 2011 by the Second Small Towns Water Supply and Sanitation Project and updated by Third Small Towns Water Supply and Sanitation Sector Project in 2015. The Design Guideline was instrumental in assisting appropriate designs and adaptation.

As the urban projects are expanded into more urban/rural areas, a need for updating the design guidelines with climate resiliency, District Metering Area (DMA) for supply network and Supervisory Control and Data Acquisition (SCADA) inclusiveness for Smart Water Management System became necessary so as to enable planners and designers for uniformity and consistency in all such projects.

This Design Guidelines will not only contribute to mainstreaming design processes but also enhance the cost efficiency and sustainability of WASH investments with emerging advanced technologies as envisaged.

This Design Guideline will be useful for Projects under UWSSP and DWSSM, staff of DWSSM, Water Supply and Sanitation Users' Committee, Design Engineers, Contractors, technical professionals and private sector engaged in the planning, design and implementation of Water Supply and Sanitation projects in urban/rural areas of Nepal.

Many professionals, with significant knowledge and experience in the planning, designing, implementing, monitoring and evaluation of WASH projects in urban/rural areas of Nepal, have contributed their time and effort in the preparation, revision and completion of the Design Guidelines. The contribution from the professionals in specific areas is duly acknowledged.

Shalik Ram Paudel
Project Director
Urban Water and Sanitation (Sector) Project,
Project Management Office (PMO).

Phone No.:977-1-4422231, 4412348, Fax No.:977-1-4413280
Email:info@uwssp.gov.np, info@uwssp.gov.np

Table of Content

Page

List of Tables

List of Figures

List of Annexes

LIST OF ACRONYMS

ADB	Asian Development Bank
DEDR	Detail Engineering Design Report
DMA	District Metering Area
DSMC	Design Supervision and Management Consultant
DWSSM	Department of Water Supply and Sewerage Management
DEWATS	Decentralized Waste Water Treatment System
EIA	Environmental Impact Assessment
EMP	Environmental Management Plan
EPA	Environment Protection Act
EPR	Environment Protection Regulation
GI	Galvanized Iron
HDPE	High Density Polyethylene Pipe
GoN	Government of Nepal
IEE	Initial Environmental Examination
IEC	Information Education and Communication
ICG	Implementation Core Group
LPCD	Liter Per Capita Per Day
KUKL	Kathmandu Upatyaka Khane Pani Limited
MOWS	Ministry of Water Supply
NGO	Non-Government Organization
NRW	Non-Revenue Water
ODF	Open Defecation Free
OHT	Over Head Tank
PAM	Project Administration Manual
PE	Polyethylene Pipe
PMO	Project Management Office
PPTA	Project Preparation Technical Assistance
PPM	Parts Per Million
PP	Procurement Plan
QA/QC	Quality Assurance / Quality Control
GRVT	Ground Reservoir Tank

UWSSP	Urban Water Supply and Sanitation (Sector) Project
TDF	Town Development Fund
TM	Transmission Main
TOR	Term of Reference
WUSC	Water Users and Sanitation Committee
WUA	Water Users Association
SDG	Sustainable Development Goal
NPC	National Planning Commission
IPP	Indigenous People Plan
DDR	Due Diligence Report
SPS	Safeguards Policy Statement
WSSP	Water Supply and Sanitation Project
WASH	Water Sanitation and Hygiene
TSTWSSP	Third Small Towns Water Supply and Sanitation Sector Project
PCR	Project Completion Report
OBA	Output Based Aid
SCADA	Supervisory Control and Data Acquisition
hSCADA	Hydraulically Supervisory Control and Data Acquisition
SMS	Source Management System
EFM	Electromagnetic Flow Meter
AMS	Air Management System
OMS	Outlet Management System
VTC	Village Transfer Chamber
AMR	Automatic Meter Reader
PFCMD	Pressure Flow Control Metering Device
RAM	Random Access Memory
GIS	Geographic Information System
NCC	Network Control Center
RTU	Remote Terminal Unit
PLC	Programmable Logic Controller
FCU	Field Control Unit
GRP	Glass Fiber Reinforced Plastic Pipe
DWC	Double Wall Corrugated Pipe

1. GENERAL BACKGROUND

With a view to streamline the design process and reporting of the Design, Monitoring and Supervision Consultants (DSMC) under the Urban Water Supply and Sanitation (Sector) Project (UWSSP), these guidelines have been prepared. The Guidelines are primarily aimed at DSMCs (and their staff), technical personnel of, DWSSM/PMO. The Design Guidelines focuses on planning and design of the water supply and Sanitation components for each project. Other sections include wastewater and sanitation, financial and economic assessment, environmental and social safeguards and related topics.

2. WATER SUPPLY COMPONENT

2.1 Technical Design Issues

There have been considerable discussions on various technical design issues in the water and sanitation sector in Nepal. These issues have ranged from establishing basic design parameters to appropriate technical alternatives available today. This write-up has been done to bring some clarity in the basic technical approach, as it can have direct impacts on the Engineering Design and estimates for various selected towns for the Feasibility Study and subsequent Detailed Engineering Design. These technical matters and other relevant issues are discussed here to optimize and rationalize the entire technical design and estimation process. Review of several output reports and design guidelines prepared earlier has helped in understanding the existing scenario and developing this approach for the engineering design of water supply and sanitation systems under the Urban Water Supply and Sanitation Project.

The desired service level for the selected towns should be better than the present service level both in terms of quantity and quality. This would mean fixing the level of service in terms of amount /quantity of water needed per capita, 24 hours continuous flow (reliability), quality of water and accessibility. In order to proceed for design, the guidelines developed by UWSSP have been adopted. Review of available documents and the desire of the users show the following desirable service level indicators:

- The pipeline network is divided into various DMAs to bring equity on the entire distribution system. One DMA generally has between 450- 1500 connections depending on the Topography of the service area;
- Each DMA is controlled and metered through one or two SCADA control valves;
- Detailed investigations (e.g. hydrogeological surveys, bore tests, etc.) are carried out to confirm adequate and sustainable yield is available from the proposed source for supply of minimum 100 lpcd.
- Per capita demand between 65 – 100 lpd. 65 lpd for yard connection and 100 lpd for fully plumbed house connections;

- Continuous 24-hour supplies with a minimum of 10 m residual pressure; and
- Quality of water confirming to the basic National Drinking Water Quality Standards for drinking water.

2.2 Service Area

A major bone of contention has been the demarcation of the service area for a water system. Service area demarcation has been done covering political boundaries like the entire commune. Typically, service area demarcation is done with respect to the feasibility of the source, settlement pattern (clustering), etc. Covering political boundaries like the entire Rural Municipality is not necessarily the most cost effective approach. If such compulsions are there then the service area can be divided into sub- area(s) with alternate sources and technologies, whichever is feasible can be adopted.

For the present purposes, the service area should be demarcated as per the technical and financial viability of the water source of the project. This means that not 100 percent of the population within the political boundary needs to be covered by the piped system. Non-piped options for uncovered areas can be provided. Conversely, any settlement outside the service area, but within relatively easy access to the source of the project should not be left out if it is technically and financially feasible.

2.3 Design Period

Design period refers to the duration for which a scheme will meet water demands of different water users. This time begins from the day a scheme is commissioned and operated by the users.

The design period of a water supply scheme generally depends on:

- Rate of population growth,
- Present and future settlement pattern,
- Economical life of the system components (Source/Pipe material/Structures/Pumps etc), and
- Potential for development.

Design periods for water supply systems in Nepal vary from 15 to 20 years for rural and semi-urban areas. A design period of 20 years shall be adopted for small towns. In addition, a base period of two to three years shall be adopted for the feasibility and detailed engineering design study (one year) and construction (one to two years).The year for which a water supply system is designed is known as design year. The sizes and capacities of all the water supply system components will be calculated for the design year. The year in which a water supply system is commissioned is known as base year.

2.4 Population

The total population of a town to be served by the proposed water supply system needs to be accurately surveyed. Firstly, the present population of the town needs to be established. Once this is done population that is likely to be reached at the base year and design year needs to be estimated.

The benefited population is defined as follows:

Present population	:	Population at the time of survey
Base year population	:	Population when construction is completed and water scheme is commissioned
Design population	:	Population at the end of design period.

2.5 Population Growth and Demographic Trends

The demographic data/information has to be carefully analyzed using previously, published information, primarily from the Central Bureau of Statistics (CBS). An assessment of the migration trends and its impact on the population in the selected town has to be addressed. The population growth during various censual periods also needs to be analyzed in reference to events like migration or exodus. The potentiality for growth vis-à-vis available space also needs to be assessed. It is necessary to assess the population density and growth rate in various wards / or sub-areas of the town, as some of the urban areas have very high growth rates but quick saturation points and others have high potential for growth.

The potential for growth in urbanized areas with a high existing population density has to be marked. However, other areas of the town in the fringes which are more rural in character with potential for growth as residential areas would have relatively a lower population density. It is necessary to have consultation and extensive assessment of the service area and divide the proposed service area into various service zones based on their growth trend and other related factors. In this regard the current trend in land transactions, construction of new houses, etc. can also be taken into consideration. The growth rate for the service zones should be based on the existing population density and the potential / saturated potential density. The design population should be based on the established growth rates for various service zones. As most of the service areas in selected towns have mixed rural and semi-urban characteristics, it is feasible to employ the geometric progression approach for projections of the design population.

The analysis of census population of a town sometimes may show the negative or zero growth of population in a particular areas/wards. It is likely that such areas/wards will have positive growth rate of population in future due to provision of water supply system and other developmental activities. The population growth rate in such wards/areas shall be adopted equal to the average population growth rate of the corresponding district. However, the population growth rate of zero can be adopted if the area/ward has reached the saturation population density. The negative population growth rate shall never be adopted.

The past census population, survey year population, growth rate and future population of the subproject area shall be presented in the format shown in Table 1.

Table 1: Population of Subproject Area

Wards	Census Population			Area	Growth Rate		Population			Population Density	
					Calculated	Adopted	Survey Year	Base Year	Design Year	Survey Year	Design Year
	1991	2001	2011	(km2)	(%)	(%)	2014	2017	2037	(person/km^2)	
Total											

2.6 Population Forecasting

A water supply project is planned to meet the present requirements as well as the requirements for a reasonable future period termed as the design period. As such it is essential to know the present population of the town and also forecast the population in base and design years. The population of a town in base and design years is forecasted on the basis of survey year population and adopted growth rate.

There are numerous methods of population forecasting. Geometrical increase method of population forecasting is most suitable for rapidly growing towns. In this method it is assumed that the percentage increase in population remains constant for each future year. The value of this constant percentage increase in population per year is applied to forecast the population in any future year. The future population Pn after n years is given by the following formula:

$$\boldsymbol{P_n = P\left(1 + \frac{r}{100}\right)^n}$$

Where,

Pn= future population at the end of n years

P = present population

r = average annual population growth rate (%)

The population of base and design year shall be forecasted for each subsystem/service zone of the subproject in the format as shown in Table 2

Table 2 : Population of Subsystems / Zones

Sub- system / Zone	Ward	Adopted Growth Rate	Population		
			Survey	Base Year	Design
		(%)	2014	2017	2037
A					
Sub-total					
B					
Sub-total					
C					
Sub-total					
D					
Sub-total					
Total					

2.7 Water Demand

The quantity of water required for various purposes in a town is known as water demand. The quantity of water required for a community will determine the sizes and capacities of all the components of water supply system such as intake chamber, transmission main, water treatment units, reservoir, distribution system, etc. The estimation of water demand or the quantity of water required for a community requires the following three factors to be known.

(a) Per capita demand of water or rate of demand
(b) Base and design period
(c) Population

The water demand scenario for small towns comprises of water required for domestic, institutional, commercial, industrial, firefighting and loss and wastage.

2.7.1 Domestic Demand

This includes the water which is required for use in private residences, apartment houses etc., for drinking, cooking, bathing, washing of clothes, washing of utensils, washing and cleaning of houses and residences, lawn watering, gardening and sanitary purposes such as flushing of water closets etc. The amount of domestic water depends on the living conditions of the consumers.

The existing households in most towns are predominantly "pucca" houses with pour- flush latrines. Settlements and houses in the periphery of the small towns are more rural in character, which may prefer yard connections or community tap-stands. These rural settlements along the periphery of the town are expected to be of small town character with “pucca” houses in near future due to provisions of water supply facilities and other development activities. So, the household demand shall be calculated by providing 100 lpcd for all private connections. If the community desires to have shared public stand posts, it should not be more than 1% of the total house connections. The water demand for public stand post shall be calculated taking 45 lpcd as domestic consumption. However, for pump schemes having head of more than 100 meters, 65 lpcd can be taken as demand for both private and yard connections.

Regarding temporary population (population living on rents) and its water demand, the per capita demand shall be taken at the same rate as permanent population and added into the total domestic demand.

2.7.2 Non-domestic Demand

The water demand required for institutional, commercial and industrial purposes is termed as non-domestic demand. The non-domestic water demand should be adopted as follows.

(a) *<u>Institutional Demand:</u>*It includes the water required for various institutions such as schools,campus etc. The institutional water demand shall be adopted as follows.

- 10 liters/pupil/day for day-scholars
- 65 liters/pupil/day for boarders

(b) *<u>Commercial Demand:</u>*This includes the water demand of commercial establishments such asoffices, hotels, hospitals, restaurants etc. The quantity of water required for this purpose will vary considerably with the nature and type of commercial establishments. The commercial water demand shall be adopted as follows,

- 10 liters/person/day for offices
- 500 liters/bed/day for hospitals with bed
- 2500 liters/day for hospitals without bed and health clinics
- 200 liters/bed/day for hotels with bed
- 500 - 1000 liters/day for hotels without bed
- 45 liters/seat/day for restaurants and tea stalls etc.

- 10 liters/seat/day for auditoriums

(c) *Industrial Demand:*The presence of industries in or near the town has great impact on the waterdemand. The quantity of water required depends upon the type of industry. The survey shall be carried out to determine the water demand of the existing industries and the provision foe industrial demand shall be made accordingly.

The non-domestic water demand in design year shall be calculated assuming that its growth will take place in the same proportion as population growth rate.

2.7.3 Leakage and Wastage

The water in this category is sometimes termed as unaccounted-for water. This includes the water lost due to leakage in mains, valves and other fittings, worn or damaged meters, theft of water through unauthorized water connections, and loss and waste of water due to other miscellaneous reasons. The loss of water due to all these reasons should be taken into account while estimating the total requirements of water. However, the quantity of water lost and wasted due to all these reasons being uncertain it cannot be precisely predicted.

The loss in the system as leakage and wastage shall be accounted as 10% of the total demand.

The calculation shall be done using the following formulas:

Amount of water to be lost due to L & W = 0.10 x TWD Where, TWD = total water demand

2.7.4 Fire Demand

It is the quantity of water required for fire-fighting purpose. In thickly populated areas, fires may break out and may result in severe damages if not controlled effectively. As such for almost all the small and medium size towns, provision should be made in the water supply system for meeting the demand of water for fire fighting

It is usual to provide for fire-fighting demand as a coincident draft on the distribution system along with the normal supply to the consumers. It is related as a function of population and may be computed from the following formulae:

$$\boldsymbol{Q = 100\sqrt{p}}$$

Where,

Q = the quantity of water in Cubic meter per day.

p = the population in thousands.

The water required for fire-fighting shall not be more than one lpcd.

2.7.5 Total Water Demand

The sum of domestic demand, non-domestic demand, leakage and waste and fire demand is known as total water demand. The Pipe Network System design is often performed for a conservative demand scenario, where the total domestic demand is based on fully plumbed

connections only. Therefore, it should be noted that the system demand for engineering design may vary from the water demand established for economic and financial analyses, which are based on categories of domestic demand and water sales.

The water demand shall be calculated for each subsystem/service zone of the subproject in the format as shown in Table 3.

Table 3 : Water Demand in Design Year

Subsystem/ Zone	Water Demand				
	Domestic	Nondomestic	Loss & Wastage	Fire	Total
	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)
A					
B					
C					
D					
E					
F					
Total					

2.8 Supply Duration and Consumption Pattern

Providing 24-hours supply versus an intermittent supply for limited hours has also been an issue of discussion. Sector experts are of the view that for maximizing returns on the investment, availability of water for 24 hours makes more sense. Several technical issues also argue strongly for providing a 24-hour supply. These include: smaller diameter pipes can be used (lower peak factor as more hours per day for supply of water is available), continuously pressurized pipes minimize back-pressure from subsurface water that might otherwise enter and pollute treated water in the distribution network and a higher quality of service to customers. A less costly and more reliable alternative is to store and supply water from an overhead water tank or tower. The engineering consultants need to carry out a detailed technical and cost comparison of these two approaches.

The distribution system capacity to meet the design water demand at the desired rate and time is determined by the service area consumption pattern. This consumption pattern is used to determine the balancing storage tank capacity. Therefore, it is essential to establish the type of consumption pattern. The consumption pattern of a typical town system recommended by the earlier Project can be used in calculating the storage capacity and system peaks. The recommended consumption pattern is given in Table 4.

Table 4 : Consumption Pattern

Hours	% of daily demand in the Terai	% of daily demand in the Hills
0500 – 0700	20	25
0700 – 1200	35	30
1200 – 1700	15	15
1700 – 1900	20	15
1900 – 0500	10	15

2.9 Peak Factors

The peak factor depicts the variation of the peak demand to the average demand typically in a day. The current practice for rural and semi-urban schemes is to adopt a peak factor of 3.0 for the distribution system. This is because the consumption pattern adopted as per the DWSS Guidelines states that 25 percent of the entire water demand is supplied in the morning for two hours. Considering the various practices, it is recommended that peak factors be adopted separately for each of the towns depending upon, its location and type of supply. Accordingly, the recommended peak factors for the design of the distribution system are recommended as given in Table 5.

Table 5 : Supply Type and Peak Factor

No.	Type of Town	Type of Supply	Adopted Peak Factor
1.	Terai	24 Hrs. – Continuous	2.4
2.	Hill	24 Hrs. – Continuous	3.0

2.10 Reservoir size/Location

The reservoir size shall be assessed based on the maximum cumulative deficit between supply and demand. However in cases, such as in the groundwater pumping, where cumulative supply is equal to cumulative daily demand size shall be the sum of maximum cumulative deficit and maximum cumulative surplus. The reservoir capacity generally lies between 20 to 30 % of total daily demand.

The location of reservoir tank should be possibly at/near the center of the service area for economy and uniform pressure in distribution network.

2.11 Residual Pressure and Velocity

In the small town/urban context, the minimum residual pressure to be maintained at all nodes shall be 10 meters.

Similarly, there must be minimum 0.2 m/sec cleaning velocity at the peak flow condition in all the pipe sections with the exception of dead ends serving 2-3 households.

2.12 Conceptualization

In a bid to reduce the investment and regular operating costs for the systems, costing for alternate system concepts need to be assessed. The general trend is that about 35-40 percent or even more of the investment is in the distribution network. Other major costs include the storage and treatment facilities.

In the storage part, if the topography is suitable to locate a ground reservoir on a height (this may be applicable in hilly towns) and distribute water by gravity to all the areas of concern with a minimum residual head of 10 m , then overhead tanks shall be avoided to reduce the cost of the project. However, such cases will not prevail in Terai towns. In this case an overhead tank shall be proposed.

2.13 Pipeline

The pipeline transfers water from the source to the service area. Pipelines require high investment outlay, and hence careful consideration is necessary for its design. Choosing its alignment, size and material, therefore, calls for utmost caution. Pipelines in a water supply scheme consist of transmission main and distribution pipelines.

2.13.1 Transmission Main

A pipe that feeds a storage reservoir from a source is called a Transmission Main. It is generally designed for average daily demand at the end of the design period without considering any peak factor. The justification shall be provided if design flow of transmission main adopted more than average daily demand.

2.13.2 Distribution System

Distribution system is a network of pipeline to convey the water from the service reservoir to the consumer premises. It is designed for the maximum or peak flow. The layout of the distribution system can in the form of branched system, looped system or combination of branched and looped system. This increase in the demand calls for water networks to be highly efficient as well as highly measurable.

Water distribution system is being designed as monolithic systems, for which should not face difficulties in achieving management goals, including: Non uniform Pressure, Non Uniform Velocity, Leakage and NRW prevent for which a DMA concept has emerged that will further strengthen the existing design system. Design of the distribution system is carried out on a district metering area (DMA) basis and Supervisory Control and Data Acquisition (SCADA).

District Metered Areas (DMAs, also called sectorisation) is one of the most promising methods of improving the water supply qualitatively and quantitatively. Non-Revenue Water, commonly known as NRW is a big challenge before the public bodies who supply water to a cluster of users.

The division of water distribution networks (WDNs) into district metering areas (DMAs) is a challenging issue and can be effective for analysis, planning and management purposes. This returns the actual optimal DMA design, flow meter on the conceptual cut and the position of the SCADA Control valves. As SCADA Control valves change the hydraulic paths of the system, the implementation of DMA can lead to a decrease in pressure and leakage through the WDN.

As a normal practice, water from one Elevated Storage Tank is supplied to a specific region. If one such region is broken down into multiple smaller sub-regions where we can monitor water input to and consumption in each of such sub-regions, these sub-regions will be called District Metered Areas.

If the distribution areas from the OHT/GRVT has clusters of high elevation differences, the DMAs should be separated based on the countours and supplied accordingly.

In short, district metered areas are small clusters of water users with a provision to individually monitor the water supplied and consumed. Each DMA can be isolated from others in two manners - either with use of isolation valves at its boundary or by cutting off the pipes connecting that DMA to other DMAs.

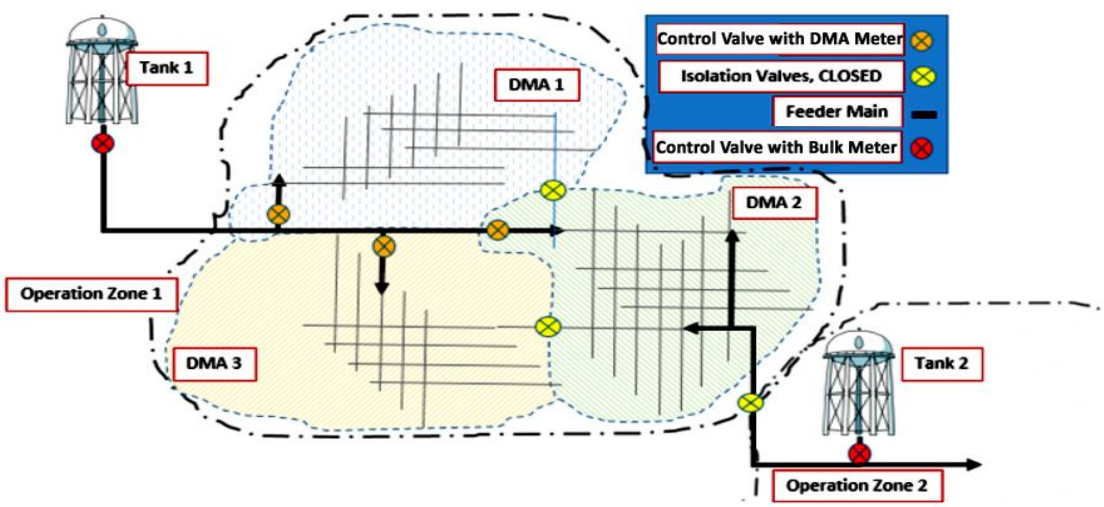


Illustration of a typical DMA

Counter to the popular belief, setting up DMAs is equally easy for urban locations where users are densely located as well as rural areas where a small number of users are sparsely spread over a large geographical area.

Methodology

Existing methods work well for small, theoretical networks, but the computing and reliability are issues when dealing with large loop system, so we need some update. Multiple factors are considered while forming DMAs in a New & existing network. Following are some of the criteria or considerations while forming DMAs:

Number of users or connections:

One DMA generally has between 450 to 1500 connections but it depended on Topography condition also. For DMAs larger than this, finding out NRW would be difficult whereas for DMAs smaller than this, the cost of monitoring and isolation equipment will go up beyond economic feasibility. The sub-DMA shall generally have a minimum of 250 connections and whose range can be 250–450 but sub-DMA design shall be designed when it's required on projects scope.

Topography:

It is recommended to use normally available topographical features such as rivers, lakes, terrain variations and even main roads to form DMAs as they will ensure ease of isolation.

Isolation and inter connectivity:

Though the DMAs should be well isolated from each other DMAs for precision in measurements, they should also be interconnected using Isolation Valves or Control Valve (initially set as closed and can be opened while responding to some emergency/ pipe break cases) so that water from one DMA can be fed into other for better distribution.

Cost of setting up DMAs:

Cost of setting up DMAs will primarily depend upon the cost of isolation, Control and metering equipment required. For the isolation purpose, standard isolation/Control valves are used whereas for the metering purpose, flow meters are used. DMAs should be formed in such a manner that, minimum number of valves and flow meters are required to achieve desired results.

Slopes and elevation:

Ideally, a DMA should comprise of uniform terrain. If a DMA has lots of terrain variations, providing water to all users at uniform pressure would be difficult and then it required extra pressure and flow regulating valves which will add extra to the project cost.

2.13.3 Pipe Materials

Much of the pressure head loss in pipelines is attributed to pipe materials used for transmission and distributing the water. Following a detailed analysis of various pipe materials used for water supply, the materials shown in Table 6 are suggested for consideration in distribution network expansion and rehabilitation subject to hydrostatic pressure and soil conditions.

Table 6 : Pipe Materials

Recommended Materials	Pipe Diameter (mm)	Remarks
uPVC	25 – 160 mm	NS 206 – 2046; IS 4985 - 2000
PE 100	15 – 315 mm	NS 40 – 2040; IS 4984 - 1978
Galvanized Iron	15 – 150 mm	NS 199 - 2046; IS : 1239 – 1990
Ductile Iron	150 – 550 mm	ISO 2531
Steel Seamless	Upto 500 mm	IS 6286

In order to use nonmetallic pipes for diameter less than 150 mm in the distribution network of the towns, the cost of commonly used pipes like HDPE and uPVC needs to be compared. However, it is essential to check the availability of technology at the local level to use uPVC.

2.14 Hydraulic Analysis

The design of the distribution system involves the determination of size of pipes to be used in the distribution system to carry the required discharge under a known pressure difference between the inlet and the exit sections of the pipe which depends upon the topography of the area. The design of the distribution requires the knowledge of the pipe hydraulics, design criteria and steps of the design process.

In the hydraulic design of a pipe the size of the pipe may be determined by using the two basic equations namely, the continuity equation and the Bernoulli's equation.

The continuity equation is as noted below:

$$\boldsymbol{Q = A \times V = \frac{\pi d2}{4} \times V}$$

In which

Q = discharge through the pipe;

A = cross-sectional area of the pipe;

d = diameter of the pipe; and

V = velocity of flow in the pipe.

For a given discharge a higher velocity of flow in a pipe requires a pipe of smaller size, and vice versa. However, a higher velocity of flow in a pipe results in excessive loss of energy or head due to friction. Similarly a much lower velocity of flow in a pipe is unsuitable, particularly for water containing suspended sediment, because it may cause the sediment to settle in the pipe.

In the Bernoulli's equation the energy or head available at the inlet section of the pipe is equated to the energy or head available at the exit section of the pipe, plus the energy or head lost and minus the energy or head added between the inlet and exit sections of the pipe. When water flows through a pipe resistance is offered to the flowing water, which results in causing a loss of energy or head. The various energy or head losses in pipes may be classified as

(i) Major losses
(ii) Minor losses

The major loss of energy or head, as water flows through a pipe, is caused by friction. It is classified as a major loss because in the case of long pipelines it is usually much more than the loss of energy or head incurred by other causes. The loss of energy or head due to friction can be determined by using either of the following formulae.

(a) Darcy Weisbach formula
(b) Hazen Williams formula
(c) Manning's formula

For the present design the Darcy-Weisbach formula can be adopted, which is one of the most commonly used formulae for determining the loss of energy or head in pipes due to friction. According to this formula the loss of head in pipes due to friction is given as

$$\boldsymbol{h_f = \frac{fLV^2}{2gd}}$$

$$\boldsymbol{h_f = \frac{fLQ^2}{2g\left(\frac{\pi d^2}{4}\right)^2 d} = \frac{fLQ^2}{12.ld^5}}$$

In which,

hf = head loss in m;

L = length of pipe in m;

d = diameter of pipe in m;

V = mean velocity of flow through pipe in m/s;

Q = discharge through the pipe in m^3/s;

g = acceleration due to gravity = 9.81 m/s^2, and

f = friction factor which is dimension less.

The value of friction factor f may be obtained by the following equation given by Colebrook and White.

$$\boldsymbol{\frac{1}{\sqrt{f}} = -2log_{10}\left[\frac{k}{3.7d} + \frac{2.5l}{R_e\sqrt{f}}\right]}$$

In which

k = roughness of the pipe material;

R_e = Reynold's number = V_d/ν; and

ν = kinematic viscosity of water

As the pipe becomes older the roughness increases due to friction. The roughness increases with time approximately in accordance with the following expression

$$\boldsymbol{k = k_0 + \alpha t}$$

In which

k_0 is the roughness of the new pipe material;

k is the roughness at any time t; and

α is rate of increase of roughness with time.

The typical pipe roughness (k) value are shown in Table 7. The value will vary depending on the manufacturer, workmanship, age and other factors. For this reason, the value given in Table 4 should be used only as a guideline.

Table 7 : Typical Pipe Roughness, k

Material	Pipe Roughness, k (mm)
Treated Water Supply Pipes (for raw water, double the values)	
Galvanized iron/Welded black pipe/Steel	0.15
Plastics, PVC-u,	0.05
PE, GRP	0.10
Ductile Iron	0.12

In addition to the friction head loss which is quite prominent in a long pipeline, a number of components in a pipe system such as valves, junctions, bends, meters etc. produces a head loss which may be substantial and should be included in the analysis of the flow distribution of the system. The need to include such losses depends on the relative importance of these losses compared to the line losses and this judgment must be made by the designer. These losses are also known as *minor losses* and can expressed as:

$$\boldsymbol{h_L = K\frac{V^2}{2g}}$$

Where,

h_L = minor head loss in m

V = the mean velocity of flow in m/s
g = acceleration due to gravity = 9.81 m/s^2

K= coefficient, the value of which depends on the type of pipe fittings.

The values of K for various types of fittings are shown in the Table 8.

Table 8: Values of K for Various Fittings

S.No.	Types of Fittings	Minor Loss Coefficient, K
1	Sudden contraction	0,30-0,50
2	Entrance, sharp well rounded	0.50
3	Elbow or bend	
	90^{o}	0,50-1.00
	45^{o}	0.40-0.75
	$22½^{o}$	0.25-0.50
4	Tee 90^{o}	1.50
5	Coupling	0.04
6	Union	0.04
7	Reducer	0.50
8	Orifice	1.00
9	Gate valve	
	Full open	2.30
	¾Open	2.60
	½Open	4.30
	¼Open	21.00
10	Globe valve	
	Full open	6.40
	½Open	9.50
11	Angle valve full open	3.00
12	Butterfly valve	
	$\theta = 5^{o}$	0.24
	$\theta = 10^{o}$	0.52
	$\theta = 40^{o}$	10.80
	$\theta = 60^{o}$	118.00
13	Check valve	
	Swing	2.00
	Disk	10.00
	Ball	70.00
14	Foot valve	15.00
15	Water meter	
	Disk	7.00
	Piston	15.00
	Rotary	10.00
	Turbine wheel	6.00

Hazen Williams Formula: Since the flow is turbulent in pipes used for water supply the friction factors depend upon the roughness of the pipe and also upon Renoulds Number which in turn depends in part upon the velocity in the pipe and its diameter. Hazen Williams formula is most used in the design of water distribution network.

$$V = K\,(C)\,R^{0.63}\,S^{0.54}$$

Where,

V = Velocity in pipe, in m/s = Discharge, Q/Area

R = hydraulic radius in m = Flow Area/Weted perimeter

S = hydraulic gradient = head loss/Length of pipe
C = a constant depending on the relative roughness of the pipe
K = a experimental cooficient, 0.849 in SI Units.

Table : Hazen Williams Cooficient for various pipes

Description of Pipes	Value of C
HDPE/PE/GRP	140 - 150
HDPE/PE/GRP old pipes including minor fittings losses	130
Cast Iron - New	130
Cast Iron – Old including fittings	100
GI - Socketed	120
Steel - welded	120
Ductile Iron	100 - 110
Ductile Iron – including minor fittings	110 - 120

$$\mathbf{HL/L = (10.659 * Q^{1.85})/(C^{1.85} * d^{4.86})}$$

Where,

HL = Head Loss, m

L = Length of Pipe, m

Q = discharge, m^3/s

C = Hazen Williams coefficient

d = diameter of pipe, m

2.15 Water Meters

Household water meters are essential for the sustainability of water systems, where collection of adequate tariff is critical. The type of water meter to be used at households should be suitable for the quality of water being delivered. Major types of water meters used for consumers are the following:

Inferential or Tangential Flow Vane Water Meters

Dry dial, magnetic drive, single or multi-jet inferential type meters meeting ISO 4064 (B) requirements are the household meters generally sought for supply and installation in many countries.

The preferred sizes are DN 15 mm and DN 20 mm for customer meters unless very large flows (for example, for a large hotel or office) are involved. DN 15 and DN 20 meters must be supplied with nuts and male threaded tails (connectors). Threads must be BSP (British Standard Pipe Thread). Preferred meters have one-piece (sealed) mechanism assemblies.

Positive Displacement or Volumetric Rotary Piston Water Meters

These type of water meters more accurate and suitable for good quality water measurement and shall comply with the requirements of ISO 4064 or equivalent.

These meters have a modular design, consisting of an outlet case and separate measuring chamber. The measuring chamber is removable and rapidly exchangeable without removing the body. Registration of flow is done reading digital counter, which directly shows the smallest measurements.

Registration is done in cubic meters. For ease and accuracy of calibration and adjustment, dials register readings of 0.05% of the nominal discharge.

All meters are provided with wire and lead seals, both to the register and to the plug covering the adjustment screws. Normal weight of the each meter is in the range between 1.1 – 1.3 kg.

A strainer is fitted to the inlet of each water meter. The strainer screen is rigid, fits snugly, easily removable and have an effective straining area at least double that of the inlet.

2.16 Supervisory Control and Data Acquisition (SCADA)

Supervisory Control and Data Acquisition (SCADA) is a control system architecture that uses computers, networked data communications and graphical user interfaces for high-level process supervisory management, but uses other peripheral devices such as Remote terminal Unit (RTU), programmable logic controller (PLC) and discrete PID controllers to interface with the process plant or machinery.

The SCADA system is a combination of hardware and software that includes in the hardware part: control valves, Pressure transmitter, flow transmitters, and electronic communication equipment and in the software part: SCADA graphical user interface pages for monitoring and controlling, server data bases, coding programming software.

Objectives of SCADA System

The main objective is to provide Inclusive, quality based and sustainable water supply service delivery through SCADA system in the propose towns with web-based Controlling and monitoring system, the system shall implement the all project component without harm to the other project components & beneficiaries.

- Monitoring: Continuous monitoring of the parameters of water quality, volume, pressure etc.
- Measurement: Measurement of variables for processing.
- Data Acquisition: Frequent acquisition of data from PLC, RTUs and Data Loggers Remote control over the system.

- NRW Data Calculation: Providing support for NRW calculations in SCADA Graphical page by continuously receiving data from field components.
  Automation of Proposed system.

Benefit of SCADA System

SCADA System is highly evolved concept, accepted widely across the globe as a standard practice. A SCADA component perform a comparison multiple activity of a traditional component which causes its initial installation cost to be high But, if we look at the benefits gained by it, then the savings from it in the long run seem to be offset by the initial investment. The use of SCADA system is also considered important for the management and operation of supply and process in water distribution sectors.

Some Important the benefits of SCADA system are following:

Tracing down NRW: NRW or non-revenue water is total water that is used or wasted but not billed. Illegally used water, leaks, unauthorized connections add to NRW. Forming DMA and periodically analyzing water consumption and associating it with revenue can give an insight into the amount of NRW. The SCADA system is an effective system for detecting and eliminating the causes of NRW, which greatly helps in detecting leakages, finding fault in the network system and real-time data collection.

Improved quality and consistency of water supply: The components of SCADA system also ensure uniform and consistent water supply to the users. Thoughtfully designed SCADA components, which also consider terrain variations, will lead to water distribution at uniform pressure.

Remote Control system: SCADA valves and pump control panels are equipped with electric remote actuators that can be easily operated via computer / mobile using the internet. Because of the SCADA system will be stored in the server, it can be operated or monitored from anywhere in the world by the authority SCADA operator. It can be stop the entire or partial water distribution system from any location in a state of emergency.

Real-Time Data Collection: The data received through the SCADA component is stored in the SCADA server data base which provides online data as storage of old and new data which can be printed or downloaded as per the requirement. SCADA system is capable of providing and storing long-term historical data's.

Smart Water Management (SWM)

The water and sanitation services are under the government distributed services in Nepal's urban areas. Recently the community owned / managed small town water and sanitation projects have come up into the picture with investment from the government, the ADB and the User's Committee at the local level at different small towns throughout the country.

By the nature of gravity flow, water intends to serve more to the connections in the low land areas than in the high land ones. Naturally, the low land area connections get water when the water taps in the high land areas are already dried up. Combined with the security of water for a 24×7 regime, interventions of different types at different points are required in terms of controlling the supplies in the forms of:

- Area wise control
- Time wise control
- Location wise control to balance the gravity benefits
- Any combination of above

Such controls are executed using different types of equipment placed at different strategic (project specific) locations and being controlled manually which is found prone to human errors and mechanical errors. These errors have been, instrumental in defying the prime water management philosophy of Equitable Distribution which in most cases is compromised with.

Together with the water management philosophy of Equitable Distribution, there are further issues in the Distribution System itself such as:

- Air management
- Flow and Pressure Management
- Surge protection
- Quantity measurement
- Reporting
- NRW Data analyzing

Smart water management is one of the important aspects in water management, especially in urban areas, where keeping records of water consumption is extremely challenging. This issue can be resolved using the SCADA system which will help water management to manage the flow of water more effectively through online and remote methods. This water management process is called Smart Water Management system.

The smart water management system is divided into a few sub-systems to control and measure of water flow, so as to achieve the objectives of the SCADA system, according to the exact conditions and requirements, meeting the scope of the project. The sub-system of SWMS is : Reservoir Management System (RMS), Outlet Management System (OMS), Pressure Managed Areas (PMA), Source Management System (SMS), Surge Protection System (SPS), Automatic meter reading (AMR), Air & Leakage Management System.

2.17 Climate Resilient Risks

The increased frequency and intensity of rainfall and flood events, engineering designs have incorporated mechanisms to inherently respond to climate risk impacts.

Climate risks: low to high

(i) increased temperature,

(ii) increased intensity of precipitation and storm events,

(iii) prolonged droughts,

(iv) floods, and

(v) rainfall triggered landslides

To safeguard subproject structures against identified climate risks engineering measures will include:

(i) Deeper aquifers as source where yields are not affected by changes in precipitation;

(ii) Pipes to be laid below ground to avoid damage during floods;

(iii) Measures for protection of sources and project related infrastructure due to landslides, erosion, or earthquakes;

(iv) Additional free-board allowance to design parameters, such as channel depth, width and slope, and increased safety easement for key facilities such as production tube wells, pump houses, and water treatment plants;

(v) Review of the interaction of reinforcing elements, such as rebar, steel, and iron, for standing water impacts on concrete curing, design life, and depth of reinforcement from concrete edge; Protection of well from flood – sufficient height of well head is kept to protect from occasional high flood/ water logging.

(vi) 10% additional capacity in drainage and water storage systems to accommodate additional run-off due to increased rainfall intensity; and

(vii) Power backup generators to ensure operation.

Additional adaptation measures incorporated in project design include

(i) Bioengineering for protection of plantations and landslide prone areas,

(ii) Water channelization in response to river flooding and glacial lake outburst floods,

(iii) Smart water management improving system resilience during periods of droughts and lowered precipitation

3. WATER QUALITY

Absolutely pure water is never found in nature. Absolutely pure water is that water which contains only two parts of hydrogen and one part of oxygen by volume and nothing else. But the water found in nature contains a number of impurities in varying amounts. Even the rain water which is absolutely pure at the instant it is formed becomes impure because it absorbs gases, minerals, dust, bacteria and other substances as its passes through the atmosphere. The rain water which flows over the ground as run off further picks up organic and suspended matter. The other portion of rain water which percolates through the ground dissolves several minerals, organic and inorganic matters while traversing through the subsurface strata. The water thus abstracted from surface and subsurface sources contains various impurities. The high amount of impurities in water may cause diseases in human beings. The high amount of impurities present in water should be removed by treating it before supplying the water to the public. However, the water is not treated to make it absolutely pure. It is not desirable to have absolutely pure water for public water supply because such water is not suitable for human body due to lack of vital minerals. The presence of minerals like iron, calcium, magnesium, etc. in small quantities in the water may be useful and good for the health of human beings. However, the presence of large quantities of these minerals will render the water suitable for human consumption. The presence of some toxic chemicals such as arsenic, cyanide, barium, cadmium, chromium, lead, etc. in water will render it unfit for human consumption. Table 9 shows the various types of impurities which occur in natural waters with their effects.

Table 9: Impurities in Water, their Causes and Effects

Type	Cause	Effects
Suspended Impurities	Bacteria	Some causes diseases
	Algae and Protozoa	Color, odor, taste and turbidity
	Clay and Silt	Turbidity
Dissolved Impurities	*(a) Salts of Calcium and Magnesium*	
	Bicarbonate	Hardness and alkalinity
	Carbonate	Hardness and alkalinity
	Sulphate	Hardness
	Chloride	Hardness
	(b) Salts of Sodium	
	Bicarbonate	Softening and alkalinity
	Carbonate	Softening and alkalinity
	Fluoride	Dental flurosis or mottled enamel
	Chloride	Taste
	(c) Metals and compounds	
	Iron oxide	Taste, red color, hardness and corrosiveness
	Manganese	Black or brown color
	Lead	Cumulative poisoning
	Arsenic	Toxicity

Type	Cause	Effects
	Barium	Toxic effect on heart and nerves
	Cadmium	Toxic and illness
	Cyanide	Fatal
	Boron	Affects central nervous system
	Selenium	Highly toxic to animals and fish
	Silver	Discoloration of skin and eyes
	Nitrate	Blue baby diseases (methemoglobinemia) in infant
	(d) Gases	
	Oxygen	Corrosiveness
	Carbon dioxide	Acidity and corrosiveness
	Hydrogen sulphide	Odor, acidity and corrosiveness
Suspended Organic Impurities	Vegetable	Color, taste and acidity
	Animal (dead)	Harmful diseases germs and alkalinity
Dissolved Organic Impurities	Vegetable	Produce bacteria
	Animal (dead)	Pollution of water and diseases germs

All these impurities may not be present in the same water and the concentrations of the impurities vary from source to source. The water obtained from the various sources should be analyzed to determine the type and amount of the various impurities present in water. This would also be assistance in planning suitable methods of treating the water to make it suitable for human consumption. The quality of water should be ascertained so that it is safe for drinking purpose. The water required for public water supply should be potable and wholesome.

It is thus essential that the quality of water supplied to the consumers be of approved standards. Therefore, National Drinking Water Quality Standards (2062) have been adopted, as prescribed in the Urban Water Supply and Sanitation Policy of 2009. This might require improved treatment of the source water. Some of the major water quality parameters to be monitored and water quality level established are listed in Table 10.

Other heavy metals and carcinogenic chemicals should not be present in the water for drinking and bacteriological contamination should be nil.

Table 10 : Water Quality Parameters as per NDWSQS 2062

S.N.	Category	Parameters	Units	Concentration Limits	Remark
1	Physical	Turbidity	NTU	5 (10)	
2		pH		6.5-8.5*	
3		Color	TCU	5 (15)	
4		Taste and Odor		Non-objectionable	
5		TDS	mg/L	1000	
6		Electrical conductivity	µs/cm	1500	
7		Iron	mg/L	0.3 (3)	
8		Manganese	mg/L	0.2	
9		Arsenic	mg/L	0.05	

S.N.	Category	Parameters	Units	Concentration Limits	Remark
10	Chemical	Cadmium	mg/L	0.003	
11		Chromium	mg/L	0.05	
12		Cyanide	mg/L	0.07	
13		Fluoride	mg/L	0.5 -1.5*	
14		Lead	mg/L	0.01	
15		Ammonia	mg/L	1.5	
16		Chloride	mg/L	250	
17		Sulphat	mg/L	250	
18		Nitrate	mg/L	50	
19		Copper	mg/L	1	
20		Total Hardness	mg/L as CaCo3	500	
21		Calcium	mg/l	200	
22		Zinc	mg/L	3	
23		Mercury	mg/L	0.001	
24		Aluminum	mg/L	0.2	
25		Residual Chlorine	mg/L	0.1-0.2*	in systems using chlorination
26	Microbiological	E. Coli	MPN/100 ml	0	
27		Total Coliform	MPN/100 ml	0 in 95% samples	

***** These values show lower and upper limits
( ) Values in parenthesis refers the acceptable values only when alternative is not available.

The water quality analysis of the proposed water source should be done on regularly so as to ascertain its quality and need of its treatment. The water analysis shall be carried out at least one time during the rainy season when its quality is at worst and heavily turbid in case of surface water.

4. WATER TREATMENT

The natural water contains various types of impurities. Absolutely pure water is never found in nature. The high concentrations of impurities in water makes the water unsuitable for drinking purpose as it may causes diseases in human beings. The treatment of water is required in such water to make it suitable for drinking purpose. Water treatment is a process of making the water suitable for the intended purpose by removing the impurities present in water. The quality of drinking water after its treatment should conform to the drinking water quality standards. During the water treatment the impurities present in water are not removed to zero but to a level which will not be harmful for intended use as per the standards. The water treated for drinking purpose should be clear, safe and wholesome. The following are some of the desired outcomes of water treatment:

- palatable: has no unpleasant taste
- safe: should not contain any pathogenic organism or chemical which is harmful
- clear: low turbidity

- colorless and odorless: aesthetically pleasing
- reasonably soft: does not consume excessive soaps or detergents
- non-corrosive: to prevent leaching from pipes or tanks
- low organic content: to prevent biological growth in pipes or tanks

Raw water may contain suspended, colloidal and dissolved impurities. The purpose of water treatment is to remove all those impurities which are objectionable either from taste and odor point of view or public health point of view. The objectives of water treatment are:

(i) to remove color, dissolved gas and murkiness of water;
(ii) to remove objectionable tastes and odor;
(iii) to remove disease producing micro-organisms so that water is safe for drinking purpose;
(iv) to remove hardness of water; and
(v) to make it suitable for a wide variety of industrial purposes such as steam generation, brewing, dyeing etc.

4.1 Treatment Processes

There are various water treatment processes. A water treatment plant utilizes many treatment processes to produce water of a desired quality. These processes can be classified as unit operations and unit processes. In unit operations, contaminants are removed by physical means. In unit processes, their removal is achieved by chemical and biological means. Many unit operations and processes can be combined to achieve a desired level of treatment. The collective arrangement of various unit operations and processes is called a process train, flow diagram or flow scheme. Each process plays an important role at various stages of the treatment train. The selection of the best water treatment alternative depends very much on the quality of the raw water and the relative cost of each alternative. The quality of the water should be analyzed to know its characteristics prior to the selection of the water treatment processes.

There are various water treatment processes. The water treatment processes should be selected depending on the type and concentration of impurities to be removed from water. The most widely used treatment processes are described below.

1. *Screening*: The purpose of screening is to remove all the large suspended and floating matters from waters. It is generally provided at the intake point.
2. *Plain Sedimentation*: The plain sedimentation is a process of settling out coarse and heavy suspended particles such as sand, silt, etc. through the force of gravity, without addition of any chemical, in a tank known as sedimentation tank or settling tank. This process removes turbidity of water as well as color, odor and taste associate with it.

3. *Sedimentation with Coagulation*: The purpose of sedimentation with coagulation is to remove the fine suspended particles and colloidal matters present in water. Coagulants are added in water to coalesce with small particles and form larger flocs which are then removed in the sedimentation tank. The complete process consists of mixing, flocculation and sedimentation.
4. *Filtration*: The process of filtration forms the most important stage in the purification of water. Filtration is a process of passing the water in a filter media, which is usually sand. It removes very fine suspended impurities and colloidal impurities that may have escaped the sedimentation tanks. In addition to this, the micro-organisms present in water are largely removed.
5. *Disinfection*: Disinfection is a process of killing the pathogenic microorganisms present in water. It is carried out to eliminate or reduce to a safe minimum limit, the remaining microorganisms, and to prevent the contamination of water during its transit from the treatment plant to the place of its consumption.
6. *Softening*: This is a process of removing the hardness of water.
7. *Miscellaneous Treatments*: These include aeration, removal of iron and manganese and removal of color, odor and taste.

Aeration: This is adopted to remove objectionable tastes and odor and also to remove the dissolved gases such as carbon dioxide, hydrogen sulphide etc. The iron and manganese present in water are also oxidizes to some extent. This process is optional and is not adopted in cases where water does not contain objectionable taste and odor or iron and manganese.

Removal of Iron and Manganese: The presence of iron and manganese causes color, odor, taste and turbidity in water. Iron and manganese are mostly found in groundwater. The dissolved iron and manganese can be precipitated out by aeration and chlorination which can then be removed by subsequent sedimentation and filtration process.

Removal of Color, Odor and Taste: The presence of color, odor and taste in water is un objectionable. The color, odor and taste are removed by plain sedimentation, sedimentation with coagulation, filtration, disinfection, aeration, removal of iron and manganese, activated carbon treatment, use of copper sulphate, etc.

Figure 1 gives the typical schematic layout of a water treatment plant for treating the surface water.

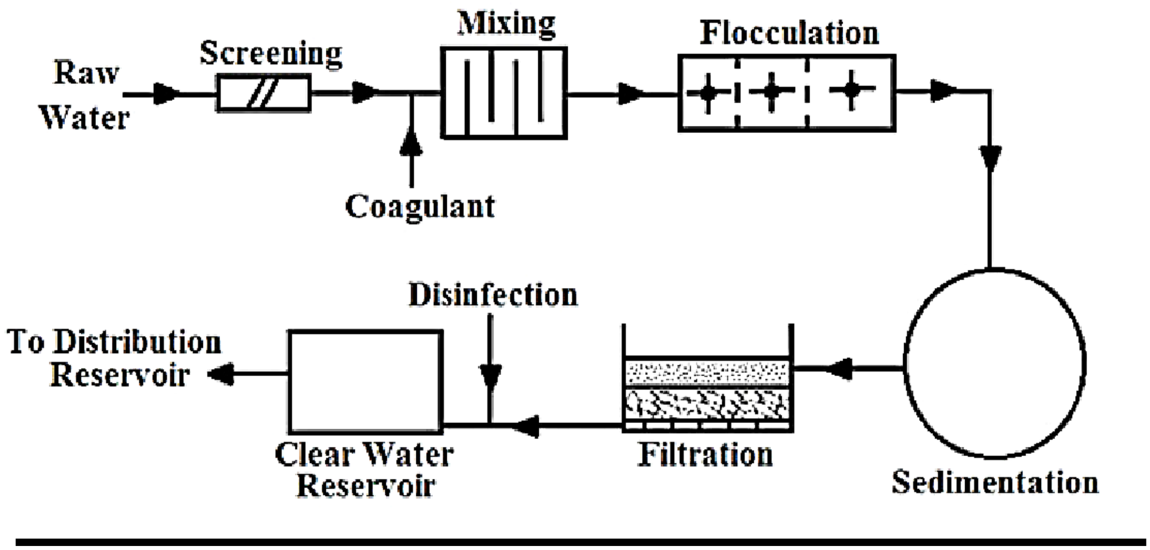


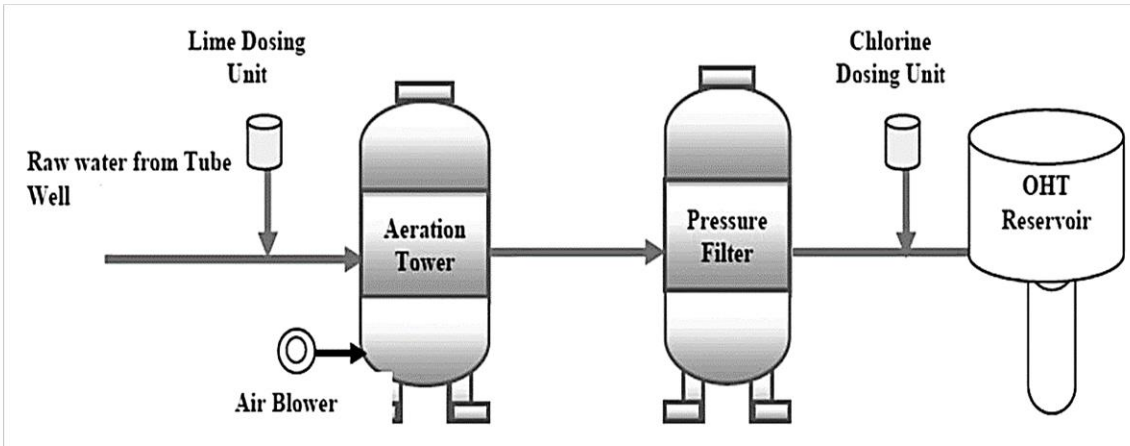
Figure 1: Schematic Layout of Water Treatment Plant (Gravity Flow)

Table 11 summarizes various water treatment processes and the impurity removal by the corresponding process.

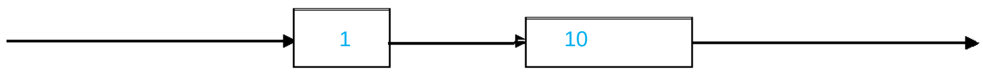
Table 11: Treatment Process and Impurity Removal

Treatment Process	Impurity Removal
Screening	Large suspended and floating matters
Plain Sedimentation	Coarse and heavy suspended particles
Sedimentation with Coagulation	Fine suspended particles and colloidal matters
Filtration	Very fine suspended impurities and colloidal, microorganisms
Disinfection Softening	Pathogenic microorganisms Hardness
Aeration	Color, odor and taste, Iron and manganese
Removal of Iron and Manganese	Iron and manganese
Removal of Color, Odor and Taste	Color, odor and taste

The various combinations of water treatment processes for surface and ground water sources are shown in Figure 2.



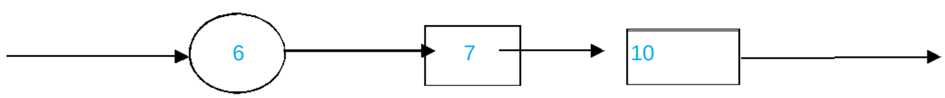
A. For Surface Water



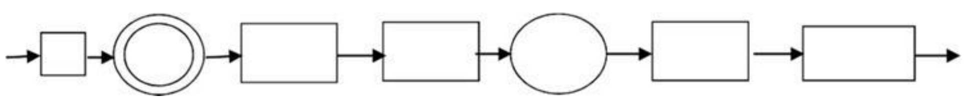
B. For Ground Water

C. For Ground Water

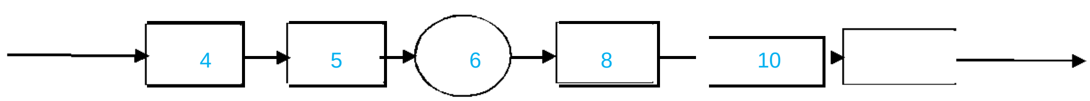
D. For Surface Water



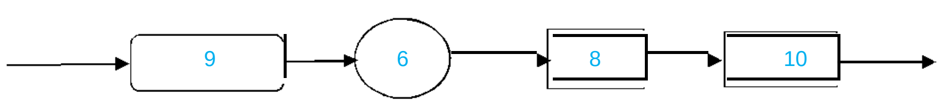
E. For Surface Water



F. For Surface Water



G. For Surface and Ground Water



1. Screening	2. Pre-chlorination
3. Aeration	4. Rapid mixing
5. Flocculation	6. Sedimentation
7. Slow Sand Filter	8. Rapid Sand Filter
9. Softening	10. Post Chlorination

Figure 2: Treatment Process Alternatives

4.2 Screening

Water derived from the surface sources, may contain suspended matter which may range from floating debris such as sticks, branches, leaves etc. Such materials if not removed will clog the pipes, form unsightly scum and sludge in the sedimentation tank and damage the pumps. Screening is a process of removing such particles by passing water through screens. Screens serve as a protective device for the remainder of the plant rather than as a treatment process. It may be located at the intake structure, raw water pumping station, or the water treatment plant itself. Screens may be of two types based on size of materials removed as follows:

(i) Bar screens; and
(ii) Fine screens

4.2.1 Bar Screens

Bar screens are intended to intercept only grosser floating materials. They are mostly in the form of steel bar grill which is either circular or rectangular in shape. The circular bars are generally of 25 mm size in diameter. The rectangular bars are generally 10 mm × 50 mm and are placed with the larger dimension parallel to the flow. The screens may be coarse or medium size depending upon the opening space between the bars. The spacing of bars may be coarse with 50 to 150 mm openings; or medium with 20 to 50 m openings. The bar screens are placed in a rectangular approach channel. Mostly, bars are kept inclined so that they can be cleaned easily with a rake. For the purposes of cleaning, they are placed on a slope of 3 to 6 vertical to 1 horizontal. The bars are sometimes kept vertical. The cleaning of inclined bar screen can be done either manually or mechanically while the vertical bar screen is cleaned mechanically.

The maximum head loss through clogged racks and screens is generally below 80 cm. Some manufactures recommend dropping the channel 150 to 300 mm across a bar screen to compensate the head loss in the racks and screens. The approaching velocity to be 0.3 to 0.6 m/s.

The head loss through unobstructed screens depends on the nature of their construction (open area, blocked area, shape of screen elements etc.), as well as the approach velocity. The head loss through screens may be obtained from manufactures' rating tables, or it may be calculated by means of the common orifice formula as given by the following expression.

$$\boldsymbol{h = \frac{1}{2gC_{d^2}}\left(V_{s^2} - V^2\right)\,\mathrm{Sin}\,\theta}$$

Where

h = head loss in m;
C_d= coefficient of discharge (generally taken as 0.6);
V_s= velocity through screens in m/s;
V = velocity in approach channel in m/s;
θ= angle of rack with horizontal; and
g = acceleration due to gravity in m/s^2.

4.2.2 Fine Screens

Fine screens are used at surface water intakes, sometimes alone and sometimes following a bar screen. It is generally made of wire mesh. The size of wire mesh should be 6 mm or more. The fine screens are generally not used in water treatment as it is frequently clogged and create difficulty in its operation and maintenance. Fine screens are sometimes used when the water is relatively clean and no other treatment except screening is employed. The usual practice in developing countries is to convey the water from the racks or bar screen to the sedimentation tank for the removal of the fine particles.

The clear water head loss through fine screens may be obtained from manufactures' rating tables, or it may be calculated by means of the common orifice formula. The velocity through screen Vs is much higher than approach velocity V. Hence, approach velocity V is generally neglected in during the calculation in fine screens.

$$\boldsymbol{h = \frac{1}{2g}\left(\frac{Q}{C_d A}\right)^2 \mathrm{Sin}\,\theta}$$

Where,

h = head loss in m;
Vs = velocity through screens in m/s;
Cd = coefficient of discharge (generally taken as 0.6);
Q = discharge through screens in m^3/s;
A = effective submerged open area in m^2;
θ = angle of rack with horizontal; and
g = acceleration due to gravity in m/s^2.

4.3 Aeration

Aeration is the process of bringing water in intimate contact with air. During aeration the gas transfer between the water and air takes place. The water absorbs the oxygen from the air and releases the carbon dioxide and other gasses from the water. Aeration in water is done to accomplish the following objectives:

(i) It removes tastes and odors causes by gases due to organic decomposition.
(ii) It increases the dissolved oxygen content of the water.
(iii) It removes hydrogen sulphide, and hence odor due to this is also removed.
(iv) It decreases the carbon dioxide content of water, and thereby reduces its corrosiveness and raises its pH value.

(v) It converts iron and manganese from their soluble states to their insoluble states, so that these can be precipitated and removed.
(vi) Due to agitation of water during aeration, bacteria may be killed to some extent.
(vii) It is also used for mixing chemicals with water.

During aeration process the gas transfer takes place between air and water. The exchange of gas from water to air or from air to water which takes place at air to water interface can be described by the following formula.

$$\boldsymbol{C_t = C_s - (C_s - C_0) exp\left\{-\left(k\frac{A}{V}t\right)\right\}}$$, for gas absorption and

$$\boldsymbol{C_t = C_s + (C_s - C_0) exp\left\{-\left(k\frac{A}{V}t\right)\right\}}$$, for gas release

Where,

C_t = actual concentration of gas in the water after given period t;
A/V = area of interface per unit volume of liquid;
C_s = gas saturation concentration;
kg = gas transfer coefficient, having dimension of velocity
C_0 = concentration gas initially present in water; and
t = aeration period

Aeration is generally done by the following main two types of aerators:

(i) Gravity aerators
(ii) Diffused aerators

4.3.1 Cascade Aerators

Cascade aerators are the simplest of the free fall aerator as shown in Figure 3. Weirs and waterfalls of any kind are cascade aerators. A simple cascade consists of a series of three to six steps of concrete or metal. Water is allowed to fall through a height of 1 to 3 meters, and due to this it comes out into close contact with air. The cascades can be either in open air, or may be in a room which has plenty of louvered air inlets. The space requirements may vary from 0.015 to 0.045 m2/m3/hr. The reduction of CO2 is usually in the range of 20 to 45% for CO2 and 35% for H2S.

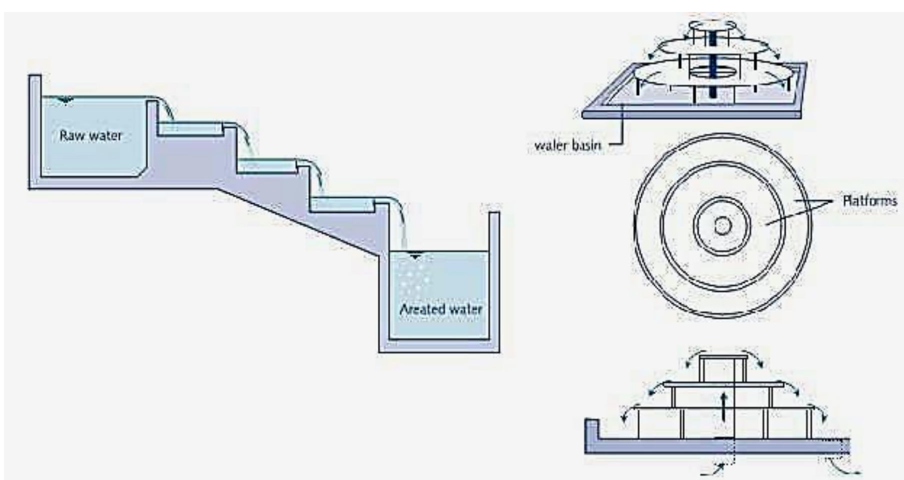


Figure 3: Cascade Aerators

4.3.2 Diffused Aerators

This is an obverse of waterfall type aerators. This type of aerator consist of a basin in which perforated pipes, porous tubes or plates are used for release of fine bubbles of compressed air in which the rise through the water being aerated. As the rising bubbles of air have a lower average velocity than the falling drops, a diffused air type provides a longer aeration time than the water fall type.

Diffused aerations may alternatively consist of tank or tower packed with pall rings, gravel or other packing materials. Tanks are normally 2 to 4 meters deep. Compressed air is injected through the system to produce fine bubbles which on rising through the water produce turbulence resulting in a continual change of exposed surface. The desired detention period varies from 10 to 30 minutes. The tank or tower is normally circular in shape. The amount of air required ranges from 0.5 to 10 m3 of air per m3 of water treated.

4.4 Chemicals Handling and Feeding

The chemicals are introduced into the water for the purpose of coagulation and flocculation, disinfection, softening, corrosion control etc. In general chemicals are added as solutions or dilute suspensions. As the water treatment process is a continuous process, the flow of chemicals is regulated and measured continuously through chemical feeders. The installation of chemical feeders promotes the uniform distribution of chemicals and eliminates wastage. The chemical feeder should be arranged and positioned in such a way that checking of dosing rate can be made at regular intervals to verify the discharge rate.

There should be at least two solution tanks for each chemical feed. The capacity of each tank should be such as to hold 8 hours requirement at the maximum demand of chemical at the design flow. A minimum free board of 0.3 m shall be provided. Adequate facilities for draining the solution tanks should be provided. The chemical solution tanks should be located in or as near the chemical storage room as possible to avoid unnecessary lifting and handling of chemicals. The tanks should be preferably located at a suitable elevation to facilitate gravity feed of the chemical solution.

It is essential to ensure that all the chemicals are dissolved before the solution is put into operation and the homogeneity of the prepared chemical solution is maintained. This can be achieved by proper mixing either by compressed air or mechanical agitation. For plants having capacities less than 2500 m3/d manual mixing may be adopted ensuring proper mixing.

The solution strength of alum which is the most widely used coagulant shall not be more than 5% for manual operation and 10% for other operations with efficient mixing. It may be desirable to dilute down to 1% prior to the addition. The chemical solution is conveyed from the solution tank to the point of application by means of chemical feed lines. These should be as short and straight as possible.

4.5 Coagulation and Flocculation

The terms 'Coagulation' and 'Flocculation' are often used indiscriminately to describe the process of removal of turbidity caused by fine suspensions, colloids and organic matter.

'Coagulation' describes the effect produced by the addition of a chemical to a colloidal dispersion, resulting in particle destabilization. Operationally, this is achieved by the addition of appropriate chemical and rapid intense mixing for obtaining uniform dispersion of the chemical.

'Flocculation' is the second stage of the formation of settleable particles (or flocs) from destabilized colloidal sized particles and is achieved by gentle and prolonged mixing.

It is common practice to provide an initial rapid or flash mixing for disposal of the coagulant or other chemicals into the water followed by slow mixing where growth of the floc takes place.

4.5.1 Choice of Coagulant

The common coagulant used in water works practice are salts of aluminum viz. filter alum, sodium aluminate and iron salts like ferrous sulphate (Copperas), ferric sulphate, ferric chloride and chlorinated copperas. The selection of aluminum or iron coagulants is largely decided by the suitability of either type or its easy availability. Alum is most widely used coagulant in water treatment because it does not cause the unsightly reddish brown staining of floors, walls and equipment which may result when iron salts are used; nor is its solution as corrosive as the ferric form of iron salts. The dissolving of ferric sulphate also offers difficulties not encountered with alum. On the other hand, ferric floc is denser than alum floc and is more completely precipitated over a wider pH range.

The choice of coagulant to be used for any particular water should preferably be based upon a series of jar tests, so planned that it will permit accurate comparison of the materials being studied under identical experimental conditions. The coagulant dose in the field should be judiciously controlled in the light of the jar test values.

4.5.2 Coagulant Dosage

Although there is some relation between turbidity of raw water and the proper coagulant dosage, the exact quantity can be determined only by trial. Even thus determined, the amount will vary with other factors such as time of mixing, pH and water temperature. The use of minimum quantity of coagulant determined to be effective in producing good flocculation in any given water, will usually require a fairly long stirring periods varying from 15 to 30 minutes in summer and 30 to 60 minutes in winter.

Addition of coagulants in excess of the determined minimum quantity may increase bactericidal efficiency. It is, however, usually more economical to use the minimum quantity of coagulant and to depend on disinfection for bacterial safety.

Alum dosages may range from 5 mg/L to 50 mg/L, depending upon the turbidity and nature of the water. At low turbidity and high dosage, Al(OH)3 is almost certain to form so that the predominant turbidity-removal mechanism is sweep coagulation. At high turbidity and lower dosages, adsorption and charge neutralization will be the predominant mechanism.

Very finely divided suspended matter is more difficult to coagulate than coarse particles, necessitating a larger quantity of coagulant for a given turbidity. The cation exchange capacity of the particles of turbidity bears a significant relationship to the success of flocculation.

4.5.3 Coagulant Aids

Coagulant aid is a chemical, which when used with main coagulant, improves or accelerates the process of coagulation and flocculation by producing quick forming dense and rapid settling flocs. Finely divided clay, fuller's earth, bentonites and activated carbon are the most commonly used materials as nuclei to floc formation. The particles may become negatively charged making them subject to attraction by the positively charged aluminum ion.

Polyelectrolytes which are polymers containing ionisable units have been used successfully as both coagulant aids and coagulants but care should be taken to guard against their toxicity. They are soluble in water, conduct electricity and are affected by electrostatic forces between their charges. Polyelectrolytes create extraordinary slippery surfaces when spilled on floor and are difficult to clean up Toxicity of any polyelectrolyte has to be checked before it can be used as coagulant or coagulant aid.

4.6 Rapid Mixing

Rapid mixing is an operation by which the coagulant is rapidly and uniformly disperses throughout the volume of water, to create a more or less homogenous single or multiphase system. This helps the formation of microflocs (Perikinetic flocculation) and results a proper utilization of chemical coagulant preventing localization of concentration and premature formation of hydroxides which lead to less effective utilization of the coagulant. The chemical coagulant is introduced at some point of high turbulence in the water. The source of power for rapid mixing to create the desired intense turbulence are gravitational and mechanical.

The intensity of mixing is dependent upon the temporal mean velocity gradient 'G'. This is defined as the rate of change of velocity per unit distance normal to a section (or relative velocity of two flow lines divided by the perpendicular distance between them) and has the dimensions of T-1 and general expressed as sec-1.

The turbulence and resultant intensity of mixing is based on the rate of power input to the water and G is calculated in terms of power input in mechanical agitated mixing by the following formula.

$$G = \sqrt{\frac{P}{\mu V}}$$

Where,

G = temporal mean velocity gradient in s^{-1};

P = total input of power in water in watts;

μV = absolute viscosity of water in N-s/m^2; and

V = Volume of water to which power is applied in m^3The rate of particulate collisions is proportional to G and sufficient gradient must be furnished to achieve the desired rate of collisions. However, G is also related to the shear forces in water; large G produces appreciable shear forces that can prevent floc formation.

The total number of particle collisions is proportional to the product of G and the detention time, t. Detention times in rapid-mix basins are from 20 to 60 sec although some have t as small as 10 sec or as long as 5 min. Velocity gradients normally range from 100 to 1000 s-1. Typical G values for the respective t are:

t = 20 s, G = 1000 s^{-1};

t = 30 s, G = 900 s^{-1};

t = 40 s, G = 790 s^{-1}; and

t > 50 s, G = 700 s^{-1}.

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds 300 m^3/hr., mechanical mixing also known as flash mixing, is desirable. Multiple units may be provided for large plants. Gravitational or hydraulic devices are simple but not flexible, while mechanical devices are flexible but require external power.

The temporal mean velocity gradient, G for gravitational or hydraulic device is given by

$$G = \sqrt{\frac{\gamma h}{\mu t}}$$

Where,

γ = specific weight of water in N/m^3;

h = head loss due to friction and turbulence in m; and

t = detention time in sec.

4.6.1 Mechanical Mixing Devices

A typical mixing basin with mechanically driven impeller or paddle is known as flash mixer. Figure 4 shows a typical flash mixer. It consists of a deep, circular or square tank which is provided with a propeller type impeller (or paddle) fixed at the lower end of a vertical shaft which is driven by an electric motor. The usual ratio of impeller diameter to tank diameter is 0.2 to 0.4 and the impeller is rotated at a speed of more than 100 rpm imparting a tangential velocity greater than 3 m/s at the tip of the impeller blade. The ratio of tank height to diameter varies 1:1 to 3:1. Further in the design of a flash mixer a detention period of 20 to 60 sec is provided.

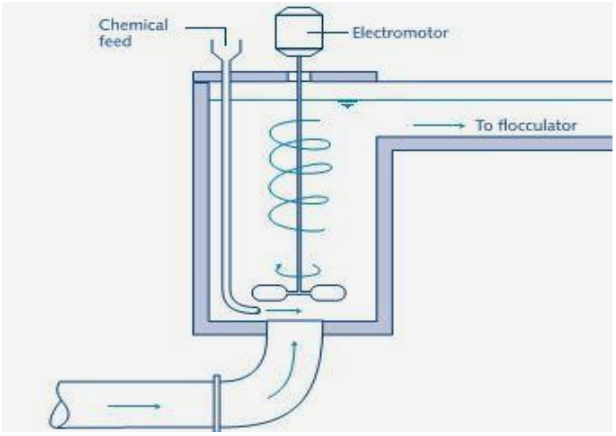


Figure 4: Flash Mixer

4.6.2 Gravitational or Hydraulic Devices

4.6.2.1 Hydraulic Jump Mixing

This is achieved by a combination of a chute followed by a channel with or without sill. The chute creates super critical flow (velocity 3 to 4 m/s), the sill defining the location of the hydraulic jump. The gentle sloping channel induces the jump. In the hydraulic jump mixing, loss of head is 0.3 m or more and detention time is brief. This device though relatively inflexible, is simple and free from moving parts. The hydraulic jump mixing is shown in Figure 5.

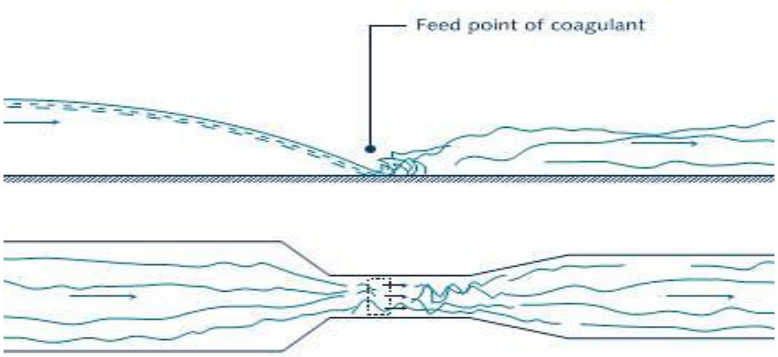


Figure 5: Hydraulic Jump Mixing

4.6.2.2 Baffled Channel Mixing

In this device the mixture of raw water and chemical is allowed to pass through a narrow mixing channel as shown in Figure 6. The channel is provided with a fairly large bed slope. Vertical baffles are provided in the channel which are projecting in an inclined position from both the sides of the channel. The angle subtended by the baffle in the channel is between 40º to 90º with the channel wall. The water flowing through the channel strikes against these baffles and creates violent agitation which causes thorough mixing of water with the chemicals. Sudden drop in hydraulic level of water over a weir can cause turbulence and chemicals can be added at this 'plunge' with the aid of diffusers. A head loss of 0.3 to 0.6 m across the weir has been reported. Similarly in pressure conduits, the chemicals can be added at the throat of a venture or just upstream of orifice located within the pipe.

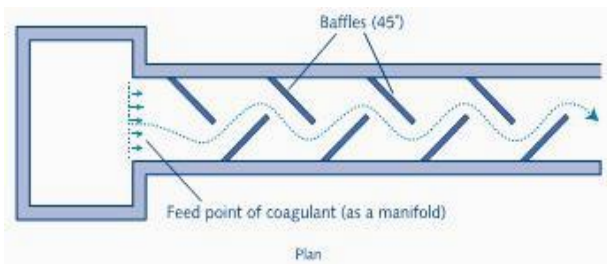


Figure 6: Baffled Channel Mixing

4.6.2.3 Overflow Weirs

Sudden drop in hydraulic level of water over a weir can cause turbulence and chemicals can be added at this 'plunge' with the aid of diffusers. A head loss of 0.3 to 0.6 m across the weir has been reported. Similarly in pressure conduits, the chemicals can be added at the throat of a venture or just upstream of orifice located within the pipe. A type of overflow weir for mixing is shown in Figure 7.

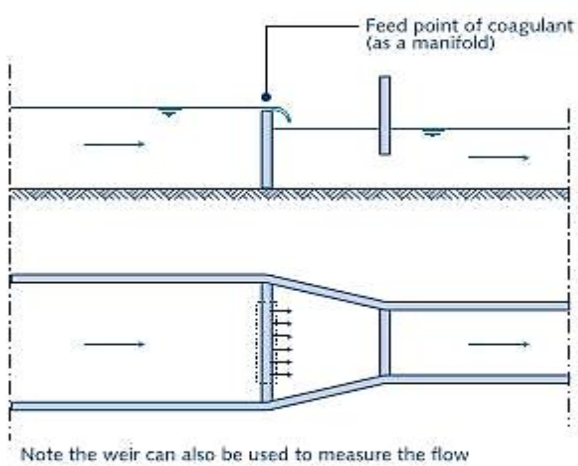


Figure 7: Overflow Weir Mixing

4.7 Flocculation

Flocculation is defined as the aggregation of destabilized particles into larger particles known as flocculent particles or “floc”. The aggregation of colloidal particles takes place in two separate and distinct phases: (1) the repulsion force between particles must be overcome; this requires that the particles be destabilized; and (2) contact between the destabilized particles must be induced so that aggregation can occur. The destabilization step is achieved by addition of chemicals to modify the electrochemistry properties on the particle surfaces. This coagulation process step is virtually instantaneous, in milliseconds to seconds, following addition of the coagulant in rapid mix tanks. The aggregation step on the other hand, requires more time for development of large flocs, by gentle stirring in the flocculation tanks. Flocculation is a slow mixing hydrodynamic process which results in the formation of large and readily settleable flocs (orthokinetic flocculation) by bringing the finely divided matter into contact with the micro flocs formed during rapid mixing. This can be subsequently removed in settling tanks and filters.

4.7.1 Design Parameters of Flocculators

The rate at which flocculation proceeds depends on physical and chemical parameters such as charges on particles, exchange capacity, particle size and concentration, pH, water temperature, electrolyte concentration, time of flocculation, size of mixing basin and nature of mixing device. The performance of flocculation can be observed by laboratory testing using Jar Test.

Since flocculation is a time rate process, the time provided for flocculation to occur is also significant factor in addition to the intensity of agitation and the total number of particles. The number of collisions is proportional to Gt where t is the detention time of the flocculation basin. The product Gt is non-dimensional and is a useful parameter for the design and operation of flocculation.

The desirable values of G in a flocculator vary from 20 70 s^{-1} and Gt from 2 to $6x10^4$ for aluminum coagulants and 1 to $1,5x10^5$ for ferric coagulants. The usual detention time varies from 10 to 50 minutes. Very high G value tend to shear flocs and prevent them from building to size that will settle rapidly. Too low G values may not be able to provide sufficient agitation to ensure complete flocculation.

Another useful parameter is the product of GT and the floc volume concentration “C” (volume of floc per unit volume of water). This parameter GCt reflects to a certain extent the contact opportunity of the particles and the values are of the order of 100.

To ensure maximum economy in the input of power and to reduce possible shearing of particles floc formation, tapered flocculation based on the concept of G is practiced. The value of G in a tank is made to vary from 100 in the first stage to 50 or 60 in the second stage and then brought down to 20 s^{-1} in the third stage in the direction of the flow.

4.7.2 Types of Flocculators

Similar to rapid mixing units, slow mixing units or flocculators can be categorized under gravitational or hydraulic and mechanical. The gravitational type uses the kinetic energy of water flowing through the plant created usually by means of baffles, while mechanical type uses the external energy which produces agitation of water.

4.7.2.1 Gravitational Flocculators

These are rectangular basins or tanks which are provided with baffle walls. The hindrance and disturbance created by the presence of baffle walls in the path of the flowing water causes vigorous agitation of water which results in through mixing of water with coagulant. Such flocculators of two types as indicated below.

(a) Horizontal flow baffled flocculator
(b) Vertical flow baffled flocculator

Figure 8 shows the plan of a horizontal or round the end type baffled flocculator. The mixture of water and coagulant after entering the basin through an inlet provided at one end of the basin, flows horizontally for a short distance and due to the presence of baffle walls it takes a turn and moves further as shown by arrows, and ultimately it flows out through an outlet provided at the other end of the basin.

Figure 9 shows the section of a vertical flow or over and under type baffled flocculator. The mixture of water and coagulant after entering the basin through an inlet provided at one end of the basin flows up and down, as shown by arrows, due to presence of vertical baffle walls projecting alternately from the roof and the floor of the basin, and ultimately it flows out through an outlet provided at other end of the basin.

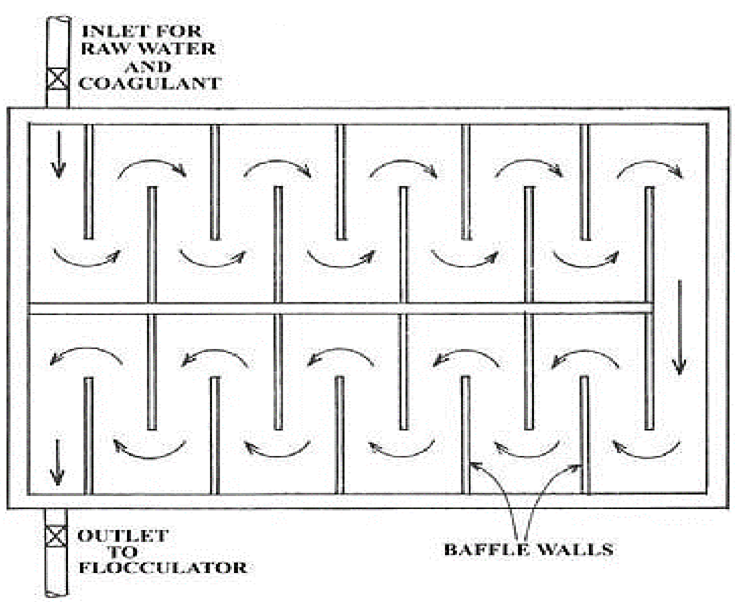


Figure 8: Horizontal Flow Baffled Flocculator

The flocculators should be properly designed to get the desired effects. The various considerations made in the design of the gravitational or hydraulic flocculators are given below.

(i) The velocity of flow in channels between successive baffles is limited to about 0.15 to 0.45 m/s.

(ii) The detention period is usually kept between 20 to 50 minutes.

(iii) To permit cleaning of the channels the distance between successive baffle walls should be at least 0.45 m.

(iv) The clear opening between the end of each baffle and the basin wall (or roof or floor as the case may be) should be about 1.5 times the distance between the successive baffle walls, subject to a minimum value of 0.675 m.

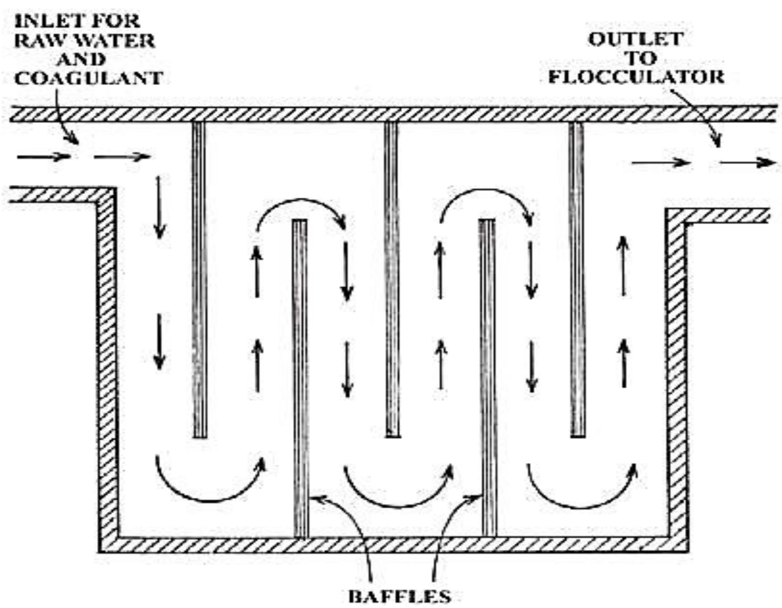


Figure 9: Vertical Flow Baffled Flocculator

As such flocculators baffled walls are generally adopted for smaller water treatment plants. For large water treatment plants mechanical flocculators are invariably used.

4.7.2.2 Mechanical Flocculators

Mechanical flocculators consist of tanks provided with paddles for stirring of water, and hence these are also known as paddle flocculators. Depending on the direction of flow of water in the tanks the mechanical flocculators may be classified as (i) longitudinal flow flocculators, and (ii) vertical flow flocculators.

A longitudinal flow flocculator consists of a rectangular tank provided with paddles revolving on a horizontal shaft as shown in Figure 10(a). A vertical flow flocculator consists of a circular tank provided with paddles revolving on a vertical shaft as shown in Figure 10(b). The paddles are rotated by an electric motor at a speed of 2 to 5 rpm.

The water coming from a mixing basin enters the flocculator tank through an inlet provided at one end of the tank and leaves it through an outlet provided at the opposite end. In a longitudinal flow flocculator both inlet and outlet are provided near the top of the tank. However, in a vertical flow tank inlet is provided at the bottom of the tank and outlet is provided near the top of the tank.

The design criteria for mechanical flocculators are as follows:

(i)	Depth of tank	:	2 to 4.5 m
(ii)	Detention period	:	10 to 40 minutes; normally 30 minutes
(iii)	Velocity of low in the flocculator	:	0.2 to 0.8 m/minute; normally 0.4 m/minute
(iv)	Total area of paddle	:	10 to 25% of the cross-sectional area of the tank
(v)	Outlet flow velocity to settling tank	:	0.15 to 0.25 m/s (to prevent settling or breaking of flocs)

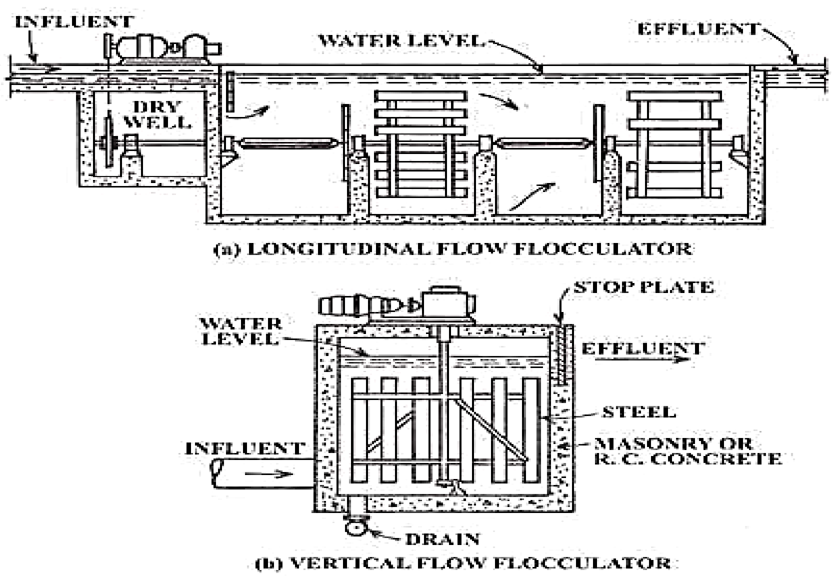


Figure 10: Mechanical Flocculators

4.8 Sedimentation

Sedimentation is the process in which water is retained in a tank or basin so that the suspended particles present in water may settle under the action of gravity. This is suitable for water containing large amounts of suspended solids of relatively large size. In water there are mainly two types of suspended solids: (i) inorganic solids having specific gravity of about 2.65, and (ii) organic solids having specific gravity in the range of 1.0 to1.4. Most of the suspended particles present in water have specific gravity greater than1 (i.e., specific gravity of water), but these are held in suspension because of turbulence of flowing water. When water enters the retaining tank it is brought to rest and there being no turbulence the suspended particles settle down and get deposited at the bottom of the tank. The tank or basin used for retention of water is known as sedimentation tank or sedimentation basin, or settling tank or settling basin. The time for which water is retained in a sedimentation tank is known as detention period or detention time or retention period. Sedimentation is one of the most commonly used unit operations in the conventional water treatment. Sedimentation (settling or clarification) is used to remove readily settling sediments such as sand and silt, coagulated impurities such as color and turbidity and precipitated impurities such as hardness and iron. When suspended solids are separated from the water by the action of natural forces alone i.e. by gravitation with or without natural aggregation, the operation is called plain sedimentation. Plain sedimentation is usually employed as a preliminary process to reduce heavy sediments loads from highly turbid raw water prior to subsequent treatment processes such as filtration. Finely divided solids and colloidal particles, which cannot be removed by plain sedimentation within commonly used detention period of few hours, are converted into settle able flocs by coagulation and flocculation and subsequently settles in sedimentation tanks.

Now a days, continuously flow sedimentation tank is generally used. In continuous flow sedimentation tank raw water is continuously admitted into the tank and allowed to flow slowly in the tank during which the particles in suspension settle down and clear water flows out continuously from the tank. The continuously flow type sedimentation tanks may be rectangular, square or circular in shape. These tanks may be classified on the basis of direction of flow of water in the tank into the following two types.

(a) Horizontal flow sedimentation tanks

(b) Vertical flow sedimentation tanks

4.8.1 Horizontal Flow Sedimentation Tanks

In a horizontal flow tank the direction of flow of water in the tank is substantially horizontal. The horizontal flow tanks may be further classified into the following two types.

(i) Rectangular tanks with longitudinal flow

(ii) Circular tanks with radial flow

(i) Rectangular tanks with longitudinal flow

Figure 11 shows a rectangular tank with baffles. The raw water enters the tank through an inlet provided on one side of the tank and after flowing slowly in horizontal direction in the tank, it passes out through an outlet provided on the opposite side of the tank. The inlet and outlet are provided at the top edge of the tank. Near the inlet and the outlet, baffles are provided to enable the flowing water to spread out evenly and thus prevent direct currents.

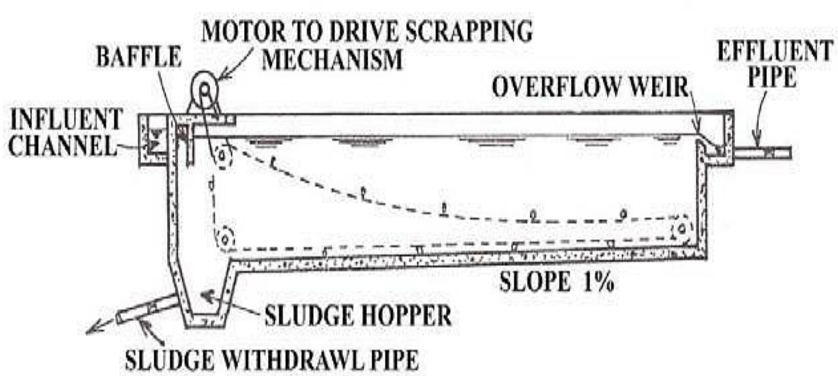


Figure 11: Rectangular Horizontal Flow Sedimentation Tank

These tanks have lengths commonly up to 30 m but larger lengths up to 100 m have also been used. The length to width ratio of these tanks should preferable be from 3:1 to 5:1. The width of these tanks is limited to 12 m. The depths commonly adopted for these tanks vary from 2.5 m to 5 m with 3 m being mostly adopted. The floor of the tank with mechanized removal of sludge is provided with a slope of 1% from the outlet end towards the inlet end where a sludge hopper with a sludge withdrawal pipe is provided. The side slopes of the sludge hopper ranges from 1.2:1 to 2:1 (vertical to horizontal). In a tank with non-mechanized (or manual) removal of sludge, the floor is provided with a cross slope of about 10% from the sided towards the longitudinal center line, and a longitudinal slope of at least 5% from the outlet end towards the inlet end where the sludge withdrawal drain is located.

(ii) Circular tanks with radial flow

The circular tanks with radial flow are of the following two types.

(a) Circular tank with central feed

(b) Circular tank with peripheral feed

Figure 12 shows the schematic diagrams of these two type of circular tanks. In a circular tank with central feed, water enters the tank at the center and leaves at its periphery. On other hand in a circular tank with peripheral feed water enters the tank from the periphery or rim and leaves at the center. Out of these two types of circular tanks the circular tank with central feed is commonly used.

Figure 13 shows a circular tank with central feed. In this tank the raw water enters continuously through a vertical inlet pipe at the center of the tank and emanates from multiple ports of influent diffuser provided at the top of the inlet pipe so that water flows radially outwards in all directions equally. A circular baffle is provided to reduce the velocity of incoming water. The water flowing slowly in the radial direction approaches a peripheral weir over which it flows into an effluent channel (or effluent launder) and finally into an effluent pipe. The sludge deposited at the bottom of the tank is continuously removed by a sludge removal mechanism. The diameter of these tanks is generally limited to about 30 m, to reduce wind effects, but tanks up to 60 m in diameter have also been used. The floors of these tanks are provided with a slope of 1 vertical to 12 horizontal from periphery towards the center.

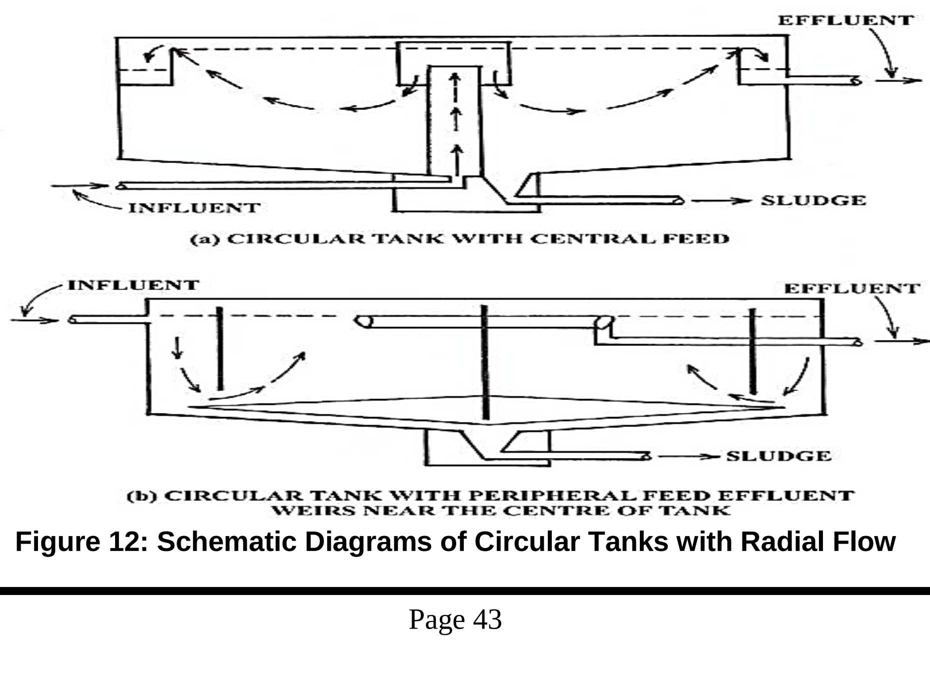


Figure 12: Schematic Diagrams of Circular Tanks with Radial Flow

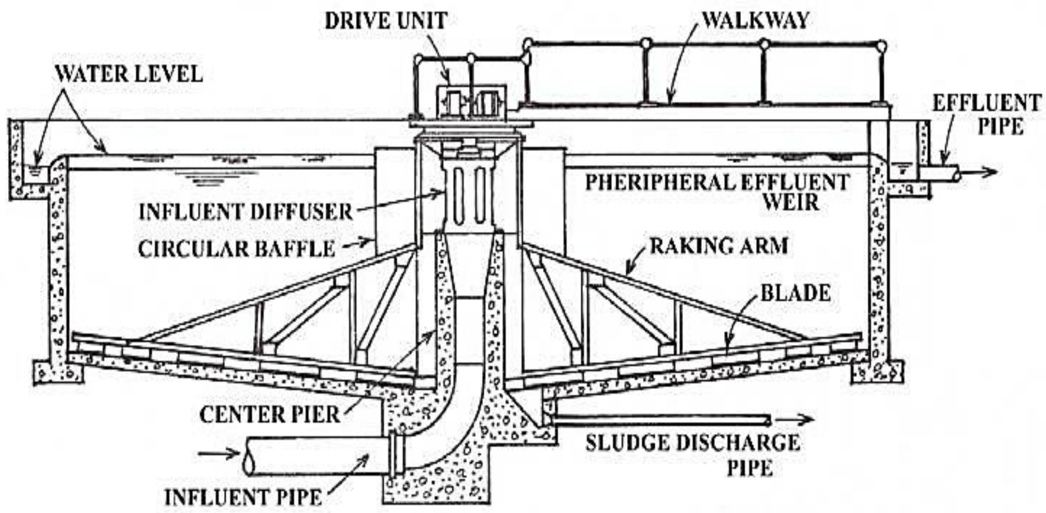


Figure 13: Circular Sedimentation Tank with Central Feed

4.8.2 Vertical Flow Sedimentation Tanks

Vertical flow tanks are square or circular in shape at the top and have hopper bottom as shown in Figure 14.

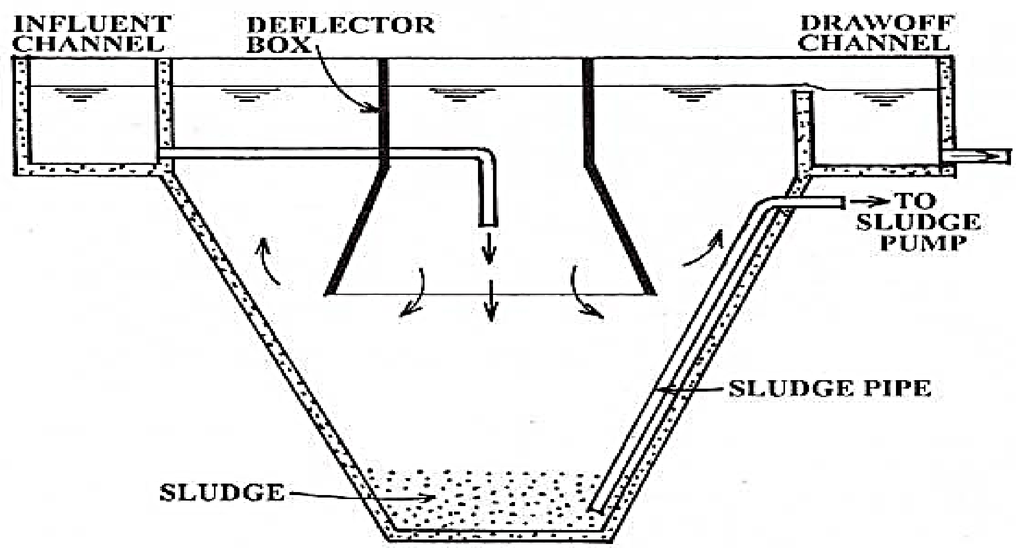


Figure 14: Vertical Flow Sedimentation Tank

The flow of water in these tanks is in the vertical direction. Water enters the tank through a centrally placed inlet pipe and by the action of a deflector box it travels vertically downwards. The sludge is collected at the bottom of the tank from where it is removed by a sludge pipe connected to a sludge pump. The clear water flows out through a circumferential weir discharging into a draw off channel.

4.8.3 Design Considerations of Sedimentation Tanks

In order to get a fairly high percentage of removal of the suspended material in a continuous flow type sedimentation tank, it is desirable that the tank is properly designed. For this it is essential to study the various design considerations for these tanks which are as follows:

(1) Velocity of flow;
(2) Relation between settling velocity of a particle and surface over flow rate;
(3) Detention period;
(4) Flowing through period;
(5) Settling tank efficiency;
(6) Inlet and outlet arrangements; and
(7) Sludge removal.

4.8.3.1 Velocity of Flow

The velocity of flow of water in sedimentation tank should be such that maximum settling of suspended particles is caused in the tank. It should remain uniform throughout the tank and it is generally in the range of 150 to 300 mm per minute.

Further it is essential that when once the particle has settled and reached the sludge zone, it should not be scoured or lifted up by the velocity of flow of water over the bed. Camp (1946) has given the following expression for the displacement velocity Vd required to start the motion of the settled particles of size d.

$$\boldsymbol{V_d = \sqrt{\frac{8\beta g}{f}(S-1)d}}$$

Where

β = 0.04 for unigranular sand; and

= 0.06 or more for non-uniform stick materials;

f = Darcy Weisbach friction factor;

= 0.025 to 0.03 for settling tanks;

S = Specific gravity of particles; and

g = acceleration due to gravity.

Thus the velocity of flow of water in the tank should be less than Vd or in other words Vd represent the maximum velocity to prevent bed uplift or scour.

The displacement velocity Vd can also be expressed in terms of settling velocity Vs by the following expression which is applicable for fine, light and flocculants solids.

The ratio of length L to depth H in a rectangular tank, or surface area A (= B × L) to cross-sectional area a (= B × H) must be kept as follows for a basin.

$$V_d = V_s \left(\frac{8}{f}\right)^{\frac{1}{2}}$$

Taking f = 0.025,

$$V_d = V_s \left(\frac{8}{0.025}\right)^{\frac{1}{2}} = 18V_s$$

A more practical relation generally used is $V_d \equiv V_s \equiv 10$

The ratio of length L to depth H in a rectangular tank, or surface area A = (B x L) to cross sectional area a = (B x H) must be kept as follows for a basin

$$\frac{L}{H} = \frac{A}{a} = \frac{V_d \times t_d}{V_s \times t_0} = \left(\frac{t_d}{t_0}\right)\left(\frac{8}{f}\right)^{\frac{1}{2}}$$

Where,

t_d= time taken by water particle to move from the inlet to the outlet (or flowing through period); and

t_0= time taken by a suspended particle to settle down (or detention period)
For an ideal tank,

$$\frac{t_d}{t_0} = 1, \therefore \frac{L}{H} = \left(\frac{8}{f}\right)^{\frac{1}{2}}$$

Taking a particle value of $\left(\frac{8}{f}\right)^{\frac{1}{2}} = \frac{V_d}{V_s} = 10$,We get $\frac{L}{H} = \frac{A}{a} = 10$

4.8.3.2 Relation between Settling Velocity and Surface Overflow Rate

Consider a rectangular sedimentation tank of length L, width B, and depth H (see Figure 15). It is assumed that the sediment is uniformly distributed as the water enters the tank at a uniform velocity V.

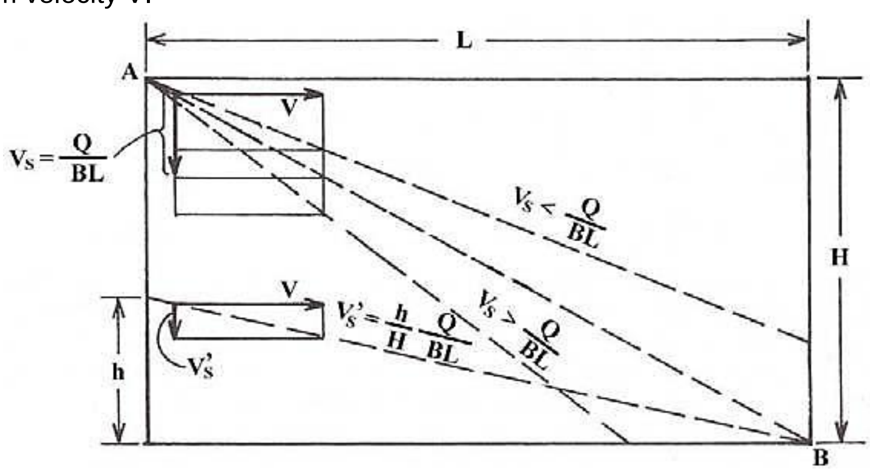


Figure 15: Ideal Sedimentation Tank

Since every discrete particle is moving with the flowing water and also tending to settle down, it possesses a horizontal velocity V (the velocity of flowing water) and a settling velocity Vs in the vertical downward direction. Thus the path of a discrete particle is given by the vector sum of the flow velocity V and its settling velocity Vs as shown in Figure 15. It is assumed that all the particles whose paths of travel are above the line AB will pass through the basin (i.e. these particles will not settle down in the tank). From geometric consideration it can be seen that

$$\frac{V}{V_s} = \frac{L}{H} \ or \ V_s = \frac{VH}{L}$$

By substituting the value V, we get $V_s = \frac{Q}{BH} \times \frac{H}{L} \text{ or } V_s = \frac{Q}{BL}$

Consideration of the assumed criterion for the settling of the particles indicates that all the particles with settling velocity Vs equals to or greater than Q/(BL) will settle down and will be removed.

Now if a smaller particle having settling velocity Vs' < Q/(BL) enters the tank at point A then it will not settle down in the tank. However, if this smaller particle enters the tank at some other level h as shown in Figure 5-15, then from geometric consideration

$$\frac{V}{V'_s} = \frac{L}{h} \ or \ V'_s = \frac{h}{L} V \therefore V'_s = \frac{h}{H}\left(\frac{Q}{BL}\right)$$

Again consideration of the assumed criterion for the settling of the particles indicates that all the particles entering at level h with settling velocity Vs' equal to or greater than [(h/H) × Q/(BL)] will settle down in the tank and will be removed.

It therefore follows that in the case of a sedimentation tank of given plan area (B × L) and given discharge Q, if all the particles entering the tank have their settling velocity equal to or greater than Q/(BL) then all the particles will settle down and will be removed. In the same tank if all the particles entering the tank have their settling velocity equal to half of Q/(BL), i.e. Q/(2BL), then only those particles which are lying below the mid-depth level will fulfill the required criterion for settling down and hence only 50% of the particles will settle down and will be removed. Thus if out of x0 particles of a particular size present in water x particles settle down and are removed, the ratio of removal of particles of this size, i.e. (x/x0) may be taken equal to (h/H) for the assumed uniform distribution of particles.

$$\frac{h}{H} = \frac{V'_s}{\left(\frac{Q}{BL}\right)} \ and \ hence, \frac{x}{x_0} = \frac{h}{H} = \frac{V'_s}{\left(\frac{Q}{BL}\right)}$$

The ratio (x/x0) therefore represents the removal efficiency of a sedimentation tank for the particles of same size.

The quantity Q/(BL) which is the discharge per unit of plan area of a sedimentation tank, is known as surface overflow rate (SOR), or overflow rate, or surface loading. The suface overflow rate (SOR) physically represents the slowest settling particle which is 100% removed. For a given discharge Q entering a sedimentation tank, increasing the plan area (B × L) of the lank will reduce the surface overflow rate and hence even those particles which are having lower values of their settling velocities will also settle down and will be removed. As such increase in the plan area of a sedimentation tank will increase the settling and removal efficiency of the tank. The surface overflow rate is therefore a significant parameter for the design of a continuous flow type sedimentation tank. The value of surface overflow rate normally adopted for the design of plain sedimentation tanks ranges from 15 to 30 m/day and for the design of sedimentation tanks using coagulants ranges from 20 to 33 m/day. Water loading rate at outlet >/ 250 m^3/m/day. (V-Notches used to measure weir length.

The above equations indicate that for discrete particles and unhindered settling, the settling of particles in a sedimentation tank is solely a function of surface overflow rate and is independent of the depth of the tank. This is, however, not correct in actual practice, because the settling of particles in a sedimentation tank depends on a number of factors and although the surface overflow rate has an important bearing in the design of sedimentation tanks, the depth of the tank is also important both for maintaining the velocity of flow as well as for providing suitable sludge storage space in the tank.

4.8.3.3 Removal efficiency of sedimentation tanks

The removal efficiency of a sedimentation tank for discrete particle of same size is given by the ratio of settling velocity of the particle and the surface overflow rate. Further it is evident that the surface overflow rate represents the settling velocity of the slowest particles which are 100% removed.

The raw water, however, contains discrete particles of different sizes which will have different settling velocities. Those particles which will have settling velocities equal to or greater than surface overflow rate will be entirely removed, while those which will have settling velocities less than surface overflow rate will be removed in direct proportion to the ration of their settling velocity Vs' to the surface overflow rate. Thus xs is the fraction of particles with settling velocity Vs' < the surface overflow rate, then the fraction of particles with settling velocity Vs' ≥ the surface overflow rate will be equal to (1 - xs) which will be entirely removed. Further the fraction of particles with settling velocity Vs' <the surface overflow rate which will be removed is obtained from equation shown below.

$$\frac{1}{\left(\frac{Q}{BL}\right)}\int_0^{x_s} V'_s\, dx$$

The Overall removal R of the discrete particles of different sizes and densities present in water as suspended particles is then given by

$$R = (1 - x_s) + \frac{1}{\left(\frac{Q}{BL}\right)}\int_0^{x_s} V'_s\, dx$$

For determining the second term in above equation, a curve indicating cumulative distribution of particle settling velocity is drawn by plotting fraction of particles with less than stated settling velocity against the settling velocity as shown in Figure 16.

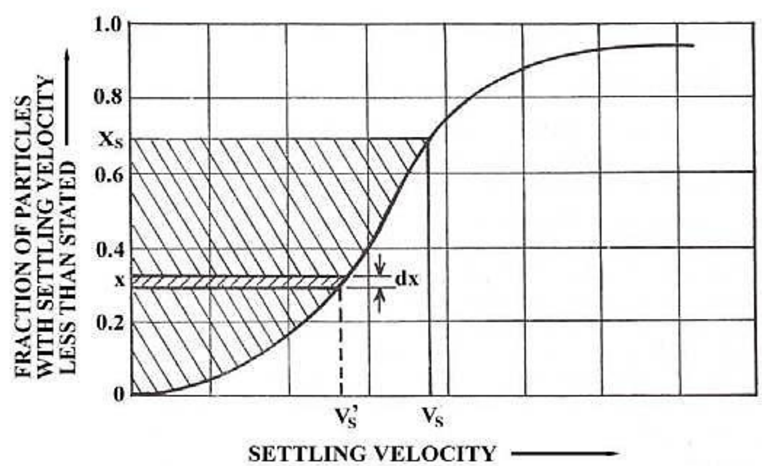


Figure 16: Cumulative Distribution of Particle Settling Velocity

The integration of the shaded portion of the curve then gives the value of the second term. The actual calculations are usually performed by graphical integration by taking finite number of points on the distribution curve, in which case the above equation is approximated as follows:

$$R = (1 - x_s) + \frac{1}{\left(\frac{Q}{BL}\right)}\sum V'_s dx$$

The above equations, however, gives only theoretical efficiency of a settling tank, because in actual practice the efficiency of a settling tank is reduced by currents induced in the tank which results in short circuiting of certain amount of water.

4.8.3.4 Detention Period

The detention period of a sedimentation tank is the theoretical time water is detained in it. In the case of a continuous flow type sedimentation tank the detention period may be considered as the theoretical time taken by a particle of water to pass from the entry to the exit of the tank. A relation between the capacity of a sedimentation tank and its detention period can be established as indicated below.

Let C be the capacity of a sedimentation tank; and Q be the discharge or rate of flow of water through the tank; then the detention period t0 is given **as** $\boldsymbol{t_0} = \frac{\boldsymbol{C}}{\boldsymbol{Q}}$

For a rectangular tank, $\boldsymbol{t_0} = \frac{\boldsymbol{LBH}}{\boldsymbol{Q}}$

For a circular tank with a bottom slope of 1 vertical to 12 horizontal

$$\boldsymbol{C = d^2(0.011d + 0.785H)}$$

$$\boldsymbol{t_0 = \frac{d^2(0.011d + 0.785H)}{Q}}$$

From the above equation it can be seen that $t_0 = \frac{H}{SOR}$, where SOR is surface overflow rate.

The value of detention period normally adopted for the design of plain sedimentation tanks ranges from 3 to 4 hours; and for the design of sedimentation tanks using coagulants ranges from 2 to 2.5 hours.

4.8.3.5 Flowing Through Period

The flowing through period is the actual time taken by the water to pass through a sedimentation tank. Theoretically the flowing through period is same as detention period, but in actual practice they vary because the displacement of the water in the tank by the incoming flow is seldom uniform. As a result, part of the water entering the tank passes out from the tank more rapidly without being detained in the tank for the intended time. In other words a part of the flowing water is short circuited. From the water which is not detained in the tank for intended time, the suspended particles present in it are not able to settle down in the tank. The settling and removal efficiency of the sedimentation tank will thus be reduced. The ratio of flowing through period and detention period which is called *displacement efficiency* is therefore a measure of the settling and removal efficiency of the sedimentation tank. In general, the displacement sedimentation tanks may vary from 25% to 50%. Flowing through period efficiency for determined with the help of dyes and chemicals such as sodium chloride, radioactive isotopes, etc.

4.8.3.6 Settling Tank Efficiency

The efficiency of a settling tank is reduced by the following currents which may be set up in the tank.

(i) *Eddy currents* set up by the inertia of the incoming water.

(ii) *Surface currents* induced due to wind in open tanks.

(iii) *Vertical convection currents* set up due to thermal gradient along the depth of the tank.

(iv) *Density currents* set up due to cold or heavy water under-running the tank and warm or light water flowing across its surface.

The current set up in the tank induces short circuiting for certain amount of water which results in reducing the efficiency of the tank in actual practice. The efficiency of a real tank affected by current induced short circuiting may be mathematical expressed as

$$\frac{y}{y_0} = 1 - \left[1 + \frac{nV_s}{\left(\frac{Q}{A}\right)}\right]^{\left(\frac{1}{n}\right)}$$

Where,

= efficiency of removal of suspended particles;

= coefficient that identifies tank performance;

= surface overflow rate for ideal settling tank; and

y/y0

n

V_s

(Q/A) = required surface overflow rate for real tank to achieve an efficiency (y/y0) for given basin performance.

The value of n are assumed as follows:

Value of n	**Performance**
0	Best
1/8	Very Good
1/4	Good
1/2	Average
1	Very poor

A well designed tank should be capable of having a volumetric efficiency of removal of suspended particles of at least 70%. Further settling tanks should be capable of giving settled water having turbidity not exceeding 20 NTU, and preferably less than 10 NTU.

4.8.3.7 Inlet and Outlet Arrangements

The water enters a sedimentation tank through an *inlet* or *influent structure* and leaves through an *outlet* or *effluent structure*. If high settling and removal efficiency is to be achieved, the inlet structure must (i) uniformly distribute flow and suspended particles over the cross-section at right angles to flow within individual tanks and into various tanks in parallel, (ii) minimize large-scale turbulence, and (iii) initiate longitudinal or radial flow. Inlet or influent structures may have different arrangements as shown in Figure 17. Each inlet opening must face a baffle so that most of the kinetic energy of incoming water will be destroyed and a more uniform lateral and vertical distribution of flow can occur. One of the satisfactory methods of attaining uniform velocity of flow is to pass the water through a training or dispersion wall with perforated holes or slots. The velocity of flow through such slots should be about 0.2 to 0.3 m/s. The diameter of the holes should not be larger than the thickness of the diffuser wall.

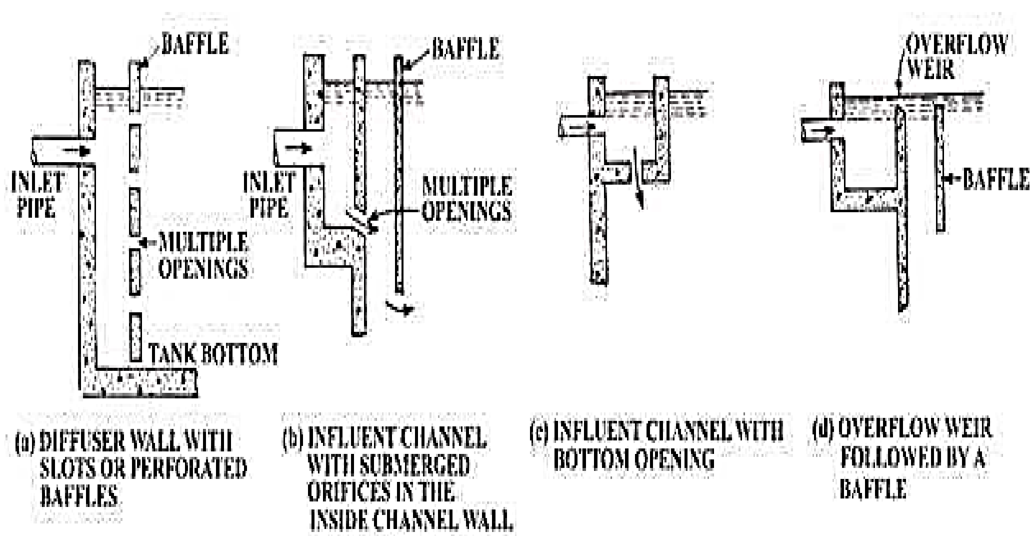


Figure 17: Inlet Arrangements

Outlet or effluent structure consists of weir, notches or orifices, effluent trough or launder and outlet pipe (see Figure 18). V-notches attached to one or both side of single or multiple troughs are normally preferred as they provide uniform distribution at low flows. The V-notches are generally placed 150 to 300 mm center to center. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent.

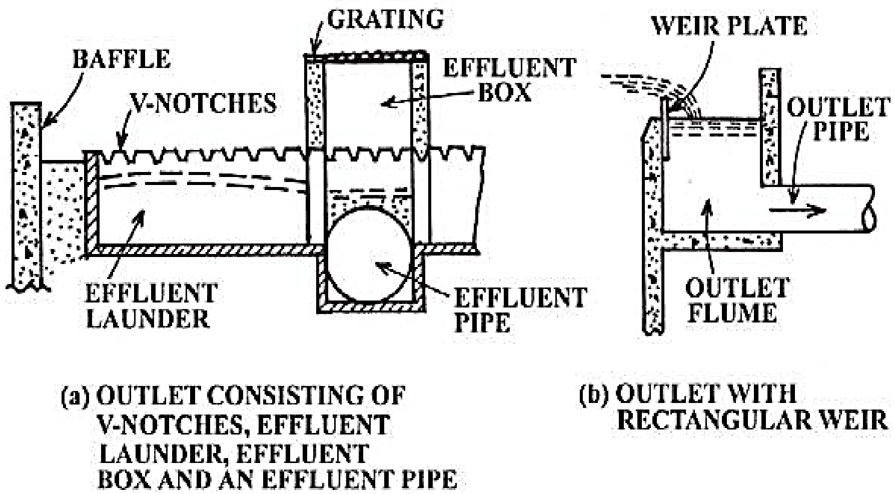


Figure 18: Outlet Arrangements

The effluent trough (or launder) acts as lateral spillways. For the design of effluent trough the following equation is generally used

$$\boldsymbol{H = \left[h^2 + \frac{2(qLn)^2}{(gb^2h)}\right]^{\left(\frac{1}{2}\right)}}$$

Where

H = water depth at upstream end of trough (or launder);
h = water depth in trough (or launder) at downstream end at distance L;
q = discharge per unit length of weir;
b = width of trough (or launder); and
n = number of sides the weir receives the flow (one or two).

In the absence of any control device, it is reasonable to assume that flow at the lower end of the trough (or launder) will be at the critical depth. Then

$$\boldsymbol{h = \left[\frac{Q^2}{(b^2)}\right]^{\left(\frac{1}{3}\right)}}$$

Where

Q is the total discharge in the trough (or launder), i.e. Q = q L.

Weir loading: Weir loading up to 300 m^3/day/m is normally adopted. However, when settling tanks are properly designed well clarified waters can be obtained at weir loading of even up to 1500 m^3/day/m.

4.8.3.8 Sludge Removal

The sludge or settled material deposited at the bottom of the sedimentation tank may be removed either manually or mechanically. With manual removal of the sludge there is a drawback that the tank needs to be shut down during the cleaning operation. However, the tanks provided with mechanical cleaning devices need not be shut down for cleaning operation and there is continuous removal of sludge from the tanks.

4.9 Filtration

Filtration is a process for separating suspended and colloidal impurities from water by passage through a porous medium or porous media. Filtration, with or without pretreatment, has been employed for treatment of water to effectively remove turbidity (e.g. silt and clay), color, microorganisms, precipitated hardness from chemically softened waters and precipitated iron manganese from aerated waters. Removal of turbidity is essential not only from the requirement of aesthetic acceptability but also for efficient disinfection which is difficult in the presence of suspended and colloidal impurities that served as hideouts for the microorganisms.

Filters can be classified according to (1) the direction of flow, (2) types of filter media and beds, (3) the driving force, (4) the method of flow rate control and (5) the filtration rate. Depending upon the direction of flow through filters, these are designated as down flow, up flow, radial flow, horizontal flow filters. Based on the media and beds, filters have been categorized into (a) granular medium filters and (b) fabric and mat filters and (c) micro strainers. The granular medium filters include single medium, dual media and multimedia (usually tri media) filters. Sand, coal crushed coconut shell, diatomaceous earth and powdered or granular activated carbon have been used as filter media but sand is widely available, cheap and effective in removing impurities. The driving force to overcome the fractional resistance encountered by the flowing water can be either the force of gravity or applied pressure force. The filters are accordingly referred to as gravity filters and pressure filters. In the fourth category are constant rate and declining or variable rate filters Lastly dependent upon the flow rates, the filters are classified as slow or rapid sand filters. Filtration of small town water supplies is normally accomplished using

i. Roughing filters;
ii. Slow sand filters;
iii. Rapid sand filters; and
iv. Pressure filters

4.9.1 Roughing Filters

Slow Sand Filtration is commonly considered an appropriate water treatment process most suitable for developing countries. The ability to significantly improve the bacteriological quality of the water without the use of any chemicals speaks in favor of this process. However, the slow sand filters are frequently overloaded with suspended solids thereby causing unacceptable short filter runs. Hence, pretreatment of the raw water is almost a necessity.

Plain sedimentation and even prolonged storage are usually not able to reduce the suspended solids concentration to the required level for successful slow sand filter operation. Destabilization of the suspension by chemical flocculation creates many operational and practical problems for a reliable application of this process in developing countries. Finally, conventional types of rapid sand filters require complicated backwash systems of a higher technical standard than that of slow sand filters. Therefore, all these processes are often inappropriate in combination with slow sand filters. Horizontal-flow Roughing Filtration might close this gap.

4.9.1.1 HRF Features

The filter is composed of a simple box filled with gravel of different sizes (from coarse to fine) as can be seen in Figure 19.

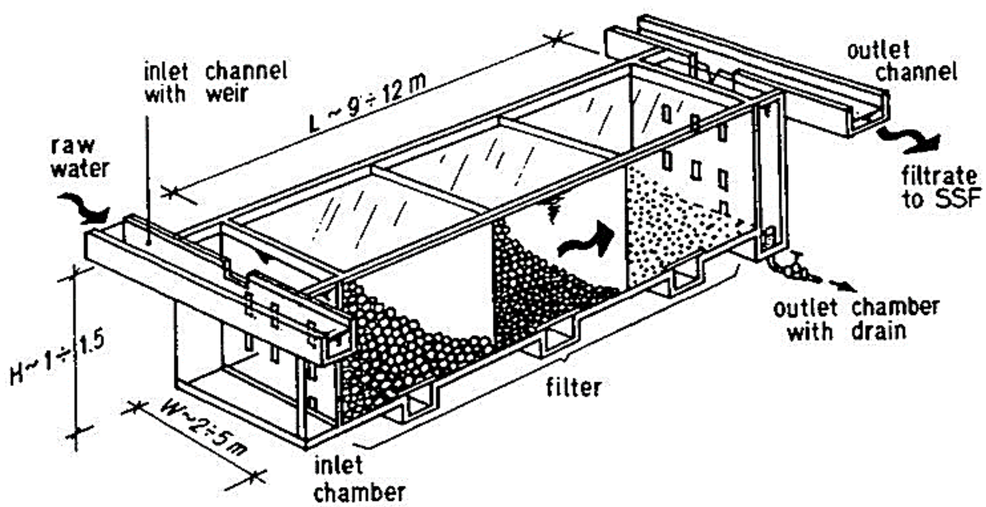


Figure 19: Horizontal Flow Roughing Filter

Horizontal-flow Roughing Filtration copies nature. The main characteristics of the process are its horizontal flow direction and the graduation of the filter material. This specific flow direction enables to construct a shallow and structurally simple filter of unrestricted length. Three to four subsequent gravel packs ranging from coarse to fine material effect a gradual removal of the solids from the water. The coarse filter material, contained in the first part of the filter, retains all the larger particles and some of the finer matter, while the last filter part with the finest filter material has to cope with the remaining smallest particles. Since the effluent of a HRF is virtually free from any solids, the standards required by SSF are easily met.

HRF is very similar to SSF. Since both filter techniques make use of natural purification processes, no chemicals are necessary to assist the treatment process. The installation of such filters requires only local resources such as construction material and manpower. Furthermore, no mechanical parts are required to operate or clean the filters. A wel1 designed filter combination will work for several months between two subsequent cleanings.

Horizontal flow roughing filters have long operational times due to their large silt storage capacity, i.e. in the order of months, similar to efficiently operating slow sand filters. Horizontal flow Roughing Filtration is used as pretreatment process prior to Slow Sand Filtration for the reduction of the raw water turbidity. The treatment combination is based on natural purification processes and therefore does not depend on any chemical supply. However, the filter units are relatively large but usually constructed with local resources. The technology is primarily meant for rural and small urban water supplies. HRF is not only used for improving the physical water quality in order to meet the slow sand filter requirements but also for removing some bacteria and viruses.

4.9.1.2 Design Guidelines

The following four design variables determine the HRF lay-out.

1) The filtration rate Vp in m/h, which is the hydraulic load in m^3/h on the filter's cross-section area in m^2
2) the individual sizes of the filter material in mm
3) the individual lengths of each filter material in m
4) the cross-section area of the filter in m^2

Unlimited filter length and simple layout are advantages of horizontal flow roughing filters. The raw water runs in horizontal direction from the inlet compartment, through a series of differently graded filter material separated by perforated walls, to the filter outlet. Filter material also ranges between 20 and 4 mm in size, and is usually distributed as coarse, medium and fine fraction in three subsequent filter compartments. To prevent algae growth in the filter, the water level is kept below the surface of the filter material by a weir or an effluent pipe placed at the filter outlet. The layout and design guideline of a horizontal flow roughing filter is presented in Figure 20.

Filtration rate in horizontal flow roughing filter ranges between 7 to 36 m/day. It has been defined hare as hydraulic load (m3/h) per unit of vertical cross section area (m2) of the filter. Filter length is dependent on raw water turbidity and usually lies within 5 to 7 m. Due to the comparatively long filter length, horizontal filter roughing filters can handle short turbidity peaks of 600 to 1,000 NTU.

Drainage facilities, such as perforated pipes, troughs or culverts, allow hydraulic filter bed cleaning. These drainage systems are placed at the filter bottom perpendicular to the direction of flow. Drainage system in the flow direction must be avoided as they could create short circuiting during normal filter operation. Hence, false filter bottom systems cannot be installed in horizontal flow roughing filters. Since most of the solids accumulate at the inlet of each filter medium, drainage facilities should be placed at the inlet of each filter compartment.

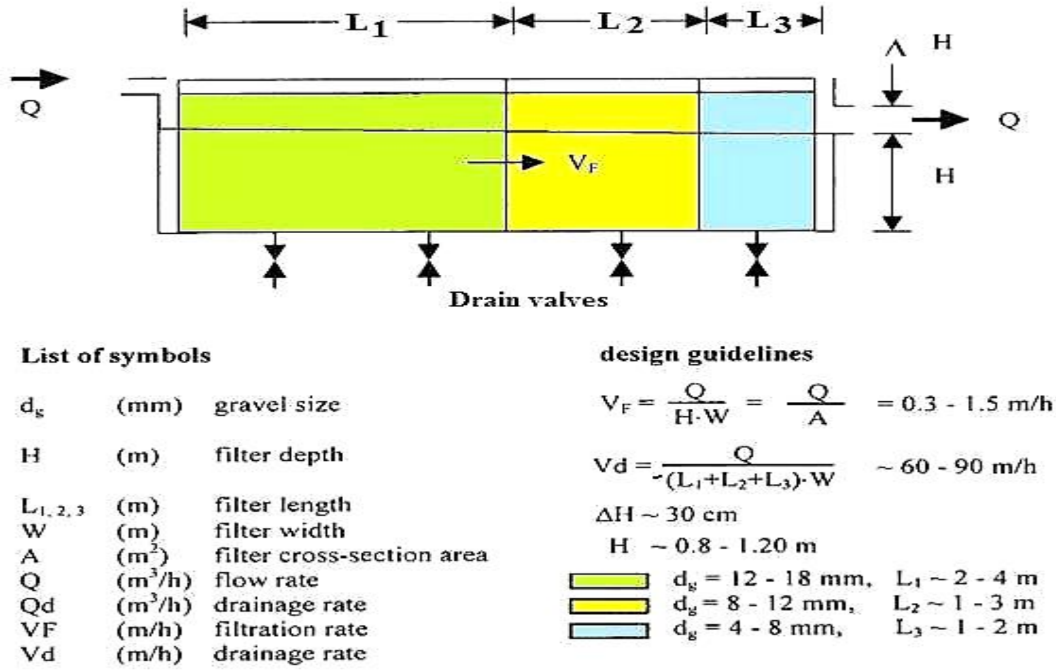


Figure 20: Layout and Design of a Horizontal Flow Roughing Filter (Wegel in 1996)

Periodic cleaning is essential for horizontal flow roughing filters. Hydraulic cleaning is carried out by fast drainage of the water stored in the filter. Drainage velocities of 60 to 90 m/h are necessary to achieve a good hydraulic cleaning efficiency. Drainage pipes of adequate size are required to achieve the recommended velocity which drains the filter within 1 to 2 minutes. Depending on the solids concentration in the raw water, regular hydraulic cleaning at intervals of every few weeks, is required to avoid deterioration of filter efficiency and development of excessive filter resistance. A reasonable drop in inlet to outlet required to be kept with free board in the chambers.

4.9.2 Slow Sand Filters

Slow sand filters can provide a single step treatment foe polluted surface waters of low turbidity (< 20 NTU) when land, labor and filter sand are readily available and equipment are difficult to procure and skilled personnel to operate and maintain are not available locally.

When raw water turbidity is high, simple pre-treatment such as its storage, sedimentation with or without primary filtration of horizontal flow roughing filters will be necessary to reduce it to within desirable limits. Chemical coagulation and flocculation have also been successfully tried to effectively pretreat turbid waters without adverse effects on filtrate quality by slow sand filtration. Impurities are removed by a combination of straining, sedimentation, biochemical and biological process during the filtration process.

A slow sand filter consists of an open box about 3.0 m deep rectangular in shape generally and made of concrete or masonry as shown in Figure 21. The box contains supernatant water layer, a bed of filter medium, an under drainage system and a set of control valves and appurtenances. A typical slow sand tank should be constructed 1 to 1.5 m below GL so that it will be 1.5 to 2 m above GL with free board and there must be manhole required for the adjustment of weir inside the outlet chamber.

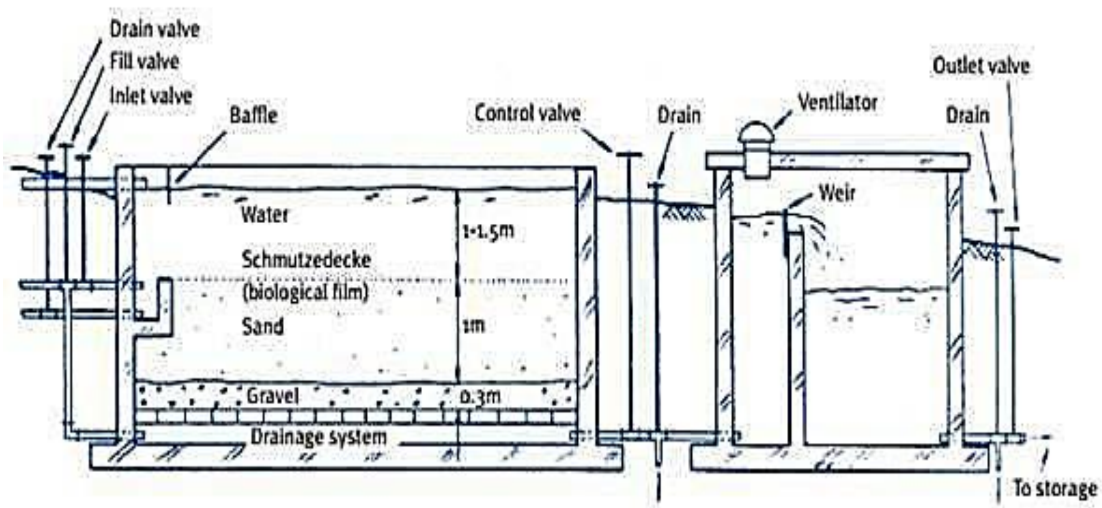


Figure 21: Typical Slow Sand Filter

The supernatant provides the driving force for the water to flow through the sand bed and to overcome frictional resistance in other parts of the system. It can also provide a storage of several hours to the incoming water before it reaches the sand surface.

The filter bed consists of natural sand with an effective size (d10) of 0.25 mm to 0.40 mm and uniformity coefficient (d60/d10) of not over 3. For best efficiency, the thickness of filter bed should not be less than 0.4-0.5 m. As a layer of 10-20 mm sand will be removed every time the filter is cleaned, a new filter should be provided with an initial sand depth of about 00.6 to 0.9 m. Re-sanding will then become necessary only once in 2-3 years.

The underdrainage system supports the sand bed and provides unobstructed passage for filtered water to leave the underside of the filter. The underdrains may be of unjointed bricks laid to form channels, perforated pipes or porous tiles laid over drains. Graded gravel to a depth of 0.2-0.3 m is placed on the underdrains to prevent the sand from entering the underdrains and ensure uniform abstraction of filtered water from the filter bed.

A system of control valves facilitates regulation of filter rate and adjustment of water level rate in the filter at the time of cleaning and back filling when the filter is put back into operation after cleaning.

4.9.2.1 Design Criteria

The design of slow sand filters depends on drinking water quality standards, raw water quality, the type and level of pre-treatment specified; and the local conditions. Great differences exist in the application of slow sand filter technology around the world. The design criteria of slow sand filter is presented in Table 12

Table 12 : Design Criteria of Slow Sand Filter

S.No.	Parameters	Unit	Criteria
1	Period of operation	hour/day	24
2	Filtration rate	m/day.	3 - 6
3	Number of filters	No.	≥ 2
4	Filter shape		Rectangular
5	Filter depth		
	Free board	m	0.2
	Supernatant water	m	1.0
	Filter sand	m	0.6 – 0.9
	Supporting gravel	m	0.3
	Under drainage system	m	0.2
	Total depth	m	2.7 – 3.10
6	Effective size of sand	mm	0.25-0.35
7	Uniformity coefficient of sand		<3
8	Size of supporting gravel	mm	3 - 60
9	Length to width ratio		≈ 2

Common grading and supporting gravel medium layer thickness:

40 – 60 mm	**120 – 200 mm**
20 – 40 mm	**80 – 120 mm**
10 – 20 mm	**80 – 120 mm**
5 – 10 mm	**60 – 80 mm**
2.5 – 5 mm	**60 – 80 mm**
Total	**400 – 600 mm**

4.9.2.2 Inlet and Outlet

(a) Inlet

The inlet structure is an important component of a slow sand filter and should be so designed and constructed as to cause minimum disturbance to the filter bed, while admitting raw water and to facilitate routine operation and maintenance. A filter needs to be cleaned periodically and this is done by lowering the water level a few centimeters below the sand bed and scraping the top layer of 10-20 mm of sand. It is found in practice that draining the water through the filter bottom takes several hours, at times 1-2 days. In order to obviate this difficulty, a supernatant drain out chamber with its top just above the sand level, has to be provided. By a proper design, the filter inlet and supernatant drain out could suitable combined in a single chamber. A typical inlet box for slow sand filter is shown in Figure 22.

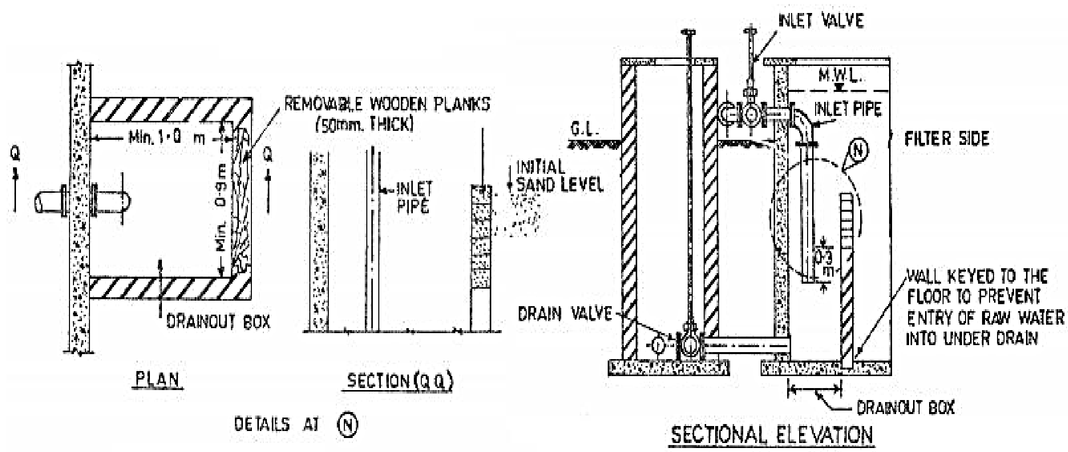


Figure 22: Inlet Box for Slow Sand Filter

(b) Outlet

The outlet superstructure incorporate means for measuring the filter flow and backfilling with clean water after sand scraping and re-commissioning of the filter. In small filters, the outlet chamber is usually constructed in two parts separated by a wall with a weir. The sill of the weir is fixed above the highest sand level in the filter bed. This makes filter operation independent of fluctuation in the clear water storage level and prevents occurrence of negative head in the filter. It also aerates the filtered water thus raising its oxygen content. To facilitate the aeration, a ventilation opening with screen is provided in the chamber. A typical outlet chamber for slow sand filter is shown in Figure 23.

4.9.2.3 Operation and Maintenance

(a) Initial Commissioning

While commissioning, a newly constructed filter is charged with water from bottom through the underdrain until it rises 0.1-0.15 m above the sand bed. This ensures expulsion of entrapped air in the filter bed and the under drainage system. The inlet valve is then gradually opened and water is admitted to the filter from top. The water is allowed to filter at approximately the normal filtration rate and the effluent is run to waste till the formation and 'ripening' of the filter skin is complete. The ripening period ends when the bacteriological analysis indicates that the effluent quality is good and can be put into distribution system

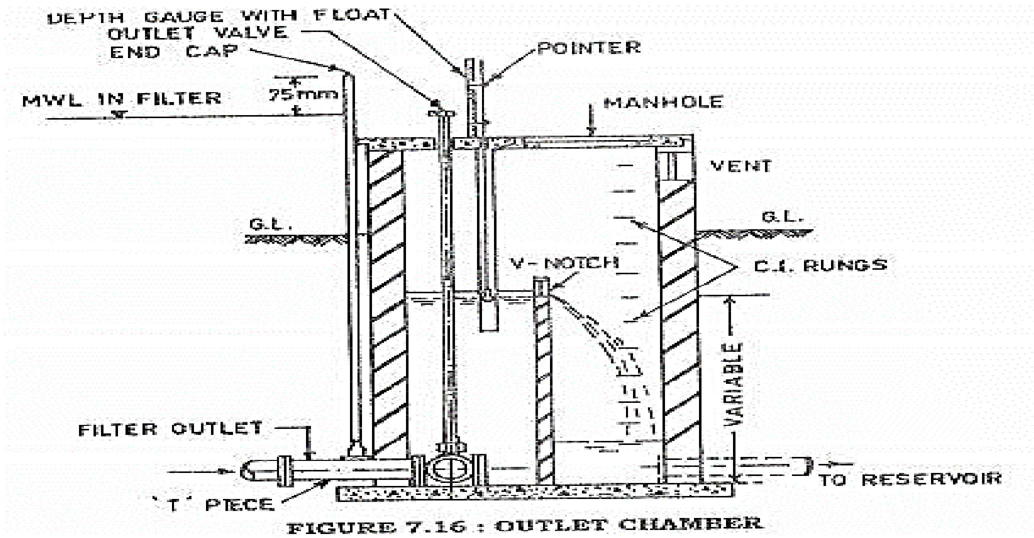


Figure 23: Outlet Chamber for Slow Sand Filter

(b) Flow Control

Controlling the flow in SSF units is necessary to maintain the proper filtration rate through the filter bed and the submergence of the media under all conditions of operation. Abrupt filtration rate increases should be avoided. Two types of flow rate control are used, outlet- and inlet-controlled flow.

In an inlet controlled filter, the rate of filtration is set by the inlet valve. Once the desired rate is reached, no further manipulation of the valve is required. At first the water level over the filter will be low but gradually it will rise to compensate for the increasing resistance of the filter skin. Once the level has reached the overflow outlet, the filter has to be taken out for cleaning. Inlet control reduces the amount of work which has to be done on the filter to just clean it. The rate of filtration will always be constant with this method and build-up of resistance in the filter skin is directly visible. On the other hand, the water is not retained very long at the beginning of the filter run, which may reduce the efficiency of treatment.

In an outlet controlled filter, which is more common, the rate of filtration is set with the outlet valve. Daily or every two days this valve has to be opened a bit to compensate for the increase in the resistance of the filter skin. The disadvantage of this method is that the outlet valve has to be manipulated on a regular basis, causing a slight variation in the rate of filtration. Thus, the operator is forced to visit the plant at least every day otherwise the output will fail. The water is retained for five to ten times as long as in the inlet controlled filter at the beginning of the filter run, which may make purification more efficient. Removal of scum will also be much simpler than with inlet controlled filtration.

(c) Filter Cleaning Control

When the filter has attained the maximum permissible head loss, it is taken out of service for cleaning. The inlet is closed and the supernatant is drained out or allowed to filter through so as to expose the sand bed. Experience has shown that filtering through takes a long time, occasionally even one or two days. Hence, lowering the water level by opening the supernatant drain out valve should be preferred. When the supernatant is drained out, the water level is lowered 10-15 cm below the top of the sand bed by opening the scour valve. Without allowing the bed to dry up, the filter is cleaned manually by removing the top layer of 2-3 cm of sand along with the filter skin. The filter is returned to service by admitting through bottom filtered water from the adjacent filter to a level of a few centimeters above the sand bed before allowing raw water from top. The removed sand is washed, dried and stored for future use.

(d) Re-sanding

Due to periodic cleaning, when the sand depth is reduced to a minimum of 0.4 m, it is necessary to make up the sand depth to the original level. This is done by replenishing with a fresh lot of sand, taking care to see that the remaining old sand is placed on top of the new sand. This avoids accumulation of dirt in the deeper layers of the filter bed and helps in quick ripening after re-sanding.

4.9.3 Rapid Sand Filters

Rapid sand filter comprises of a bed of sand serving as a single medium granular matrix supported on a gravel overlaying underdrainage system as shown in Figure 24. The distinctive features of rapid sand filtration as compared to slow sand filter include careful pretreatment of raw water to effectively flocculate the floc particles, use of higher filtration rates, coarser but more uniform filter media to utilize greater depths of filter to trap influent solids without excessive head loss and backwashing of the filter bed by reversing the flow direction to clean the entire depth of filter.

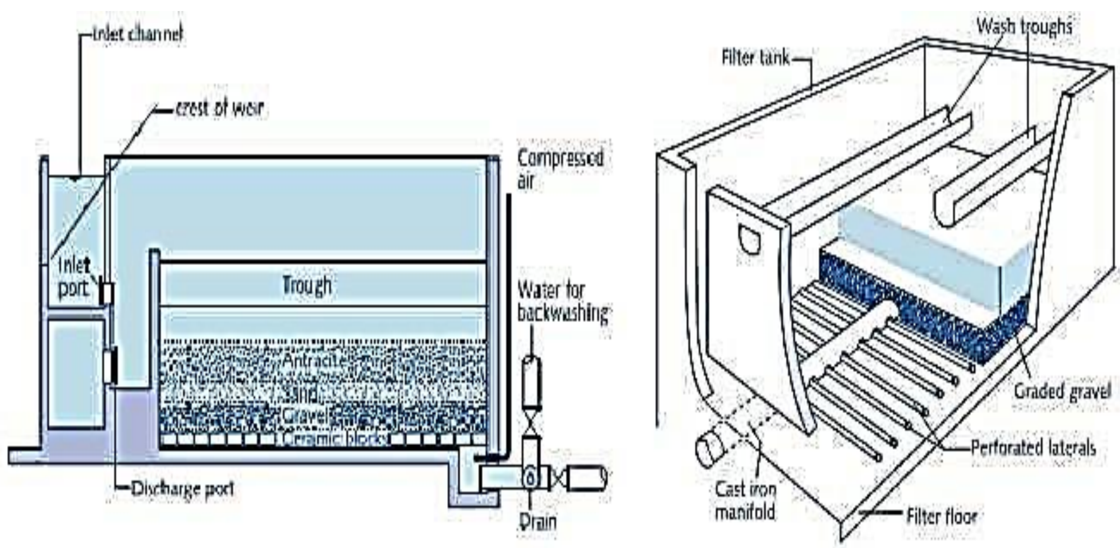


Figure 24: Rapid Sand Filter

Pretreatment of final influents should be adequate to achieve efficient removal of colloidal and suspended solids despite fluctuations in raw water quality. Pretreatment generally provided for rapid sand filtration is sedimentation with coagulation and flocculation as shown in Figure 25.

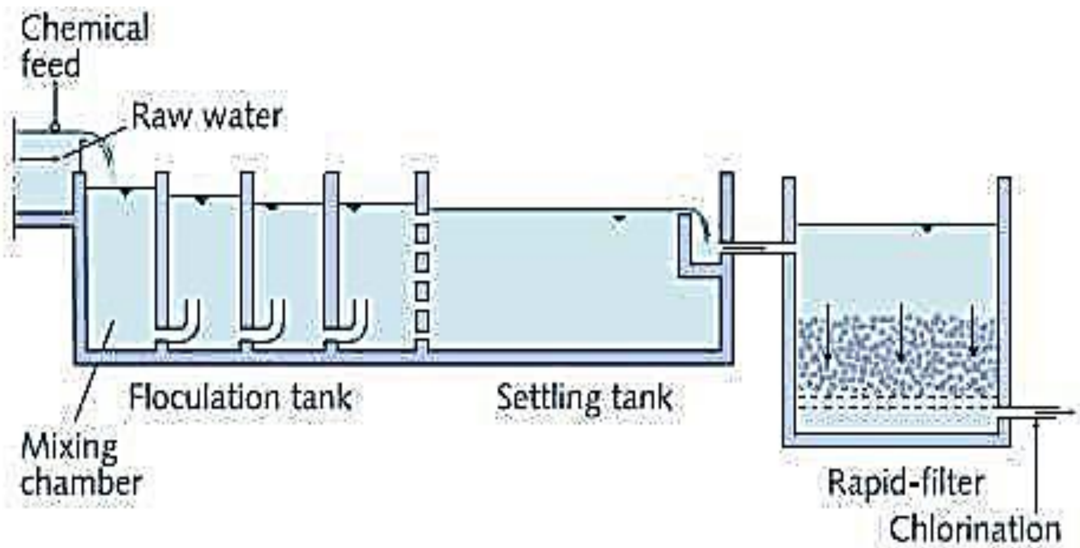


Figure 25: Rapid Sand Filter with Pretreatment

4.9.3.1 Theoretical Aspects

The overall removal of impurities from the water in rapid filtration is brought about by a combination of several different processes. The most important are straining, sedimentation, adsorption, and bacterial and biochemical processes. In rapid filtration, however, the filter bed material is much coarser and the filtration rate much higher. These factors completely alter the relative importance of the various purification processes.

The straining of impurities is not an important mechanism in rapid sand filtration due to the relatively large pores in the filter bed. Sedimentation is a significant mechanism, as in down flow filtration particles collect preferentially on the top of grains, forming caps; this is also due to the laminar flow regime. Thus, straining and sedimentation will retain far fewer impurities than in a slow sand filter. The upper filter bed layers in particular will be far less effective and there will be a deep penetration of impurities into the entire bed of a rapid filter.

By far the most important purification effect in rapid filtration is the adsorption of impurities. Although the surface of initially clean sand has a small negative electrostatic charge, this negative charge is neutralized very close to the particle by dissolved material in the water, and deposited positively charged material like aluminum or iron flocs. Thereafter, positively charged flocs encounter positively charged floc-covered sand surfaces, and electrostatic repulsion may be evident. The magnitude and effect of this is not only dependent on the electrostatic charges on the floc particle and sand surface, but also the amount and nature of the salts dissolved in the water. The more minerals the water contains, the less is the range of effect of the electrostatic forces. Most significantly, forces of Van der Waals occur at very close approach (less than 1 micro meter) of the surfaces of the particle and the grain, due to the electronic nature of the atoms and molecules of the two approaching surfaces. Such forces always attract and account for the attachment (adsorption) of flocs and other particles with an electric charge to sand grains, and existing deposits with an opposite electric charge.

In a slow sand filter the water stays in the filter bed for several hours, but with rapid filtration the water passes in only a few minutes. Accumulated organic deposits are frequently removed from a rapid filter when the filter is cleaned by backwashing. There is very little time and opportunity for any biodegradation of organic matter to develop, or for killing of pathogenic micro-organisms to take place. The limited degradation of organic matter need not be a serious drawback as the accumulated deposits will be washed out of the filter during backwashing. The poor bacteriological and biochemical activity of a rapid filter will generally be insufficient to produce bacteriologically safe water. Hence, further treatment such as slow sand filtration or chlorination will be necessary to produce water that is fit for drinking and domestic purposes.

When water containing suspended matter is applied on the top of the filter bed, suspended and colloidal solids are left behind in the granular medium matrix. Accumulation of the suspended particles in the pores and the surface of filter medium leads to build up of head loss as pore volume is reduced and greater resistance is offered to the flow of water simultaneously with the buildup of head loss to a predetermined terminal value, the suspended solids removal efficiency of successive layers of filter medium is reduced as solids accumulate in the pore space and reach an ultimate value of solids concentration as defined by operating conditions. This results eventually in breakthrough of suspended solids and the filtrate quality deteriorates. Ideally, a filter run should be terminated when the head loss reaches a predetermined value simultaneously with the suspended solids in filtrate attaining the preselected level of acceptable quality.

4.9.3.2 Design Criteria

For the design of a rapid filter, four parameters need to be selected: the grain size of the filter material; the thickness of the filter bed; the depth of the supernatant water; and the rate of filtration. To the extent possible, these design factors should be based on experience obtained in existing plants that treat the same or comparable raw water. When such experience does not exist, the design should be based on the results obtained with a pilot plant operating experimental filters. The quality of the influent water (usually pre-treated water) greatly influences the range of the design parameters. A broad indicative ranges of design criteria have been given in Table 13.

4.9.3.3 Underdrainage System

In rapid sand filters the under-drainage system serves two purposes:

(a) It collects the filtered water uniformly over the area of gravel bed.

(b) It provides uniform distribution of backwash water without disturbing or upsetting the gravel bed and filter media.

Table 13: Design Criteria of Rapid Sand Filter

S.No.	Parameters	Unit	Criteria
1	Period of operation	hour/day	23.5
2	Filtration rate	m/day	120 - 240
3	Number of filters	No.	≥ 2
4	Filter shape		Rectangular
5	Filter depth	m	≥ 0.5
	Free board		
	Supernatant water	m	0.6-2.0
	Filter sand	m	0.6-0.9
	Supporting gravel	m	0.4-0.6
	Underdrainage system	m	varies
6	Effective size of sand	mm	0.45-0.60
7	Uniformity coefficient of sand		<1.70
8	Size of supporting gravel	mm	2-60
9	Minimum surface area per unit	m^2	10
10	Maximum surface area per unit	m^2	30
11	Length/width ratio		1.25-1.35

Common grading and supporting gravel medium layer thickness:

40 – 60 mm	**120 – 200 mm**
20 – 40 mm	**80 – 120 mm**
10 – 20 mm	**80 – 120 mm**
5 – 10 mm	**60 – 80 mm**
2.5 – 5 mm	**60 – 80 mm**
Total	**400 – 600 mm**

There are various forms of under drainage systems for these filters out of which the following two systems are commonly adopted which are described below.

(a) Perforated pipe system

(b) Pipe and strainer system

(a) Perforated Pipe System

In this system, there is a central drain or manifold to which a number of laterals are attached on either side as shown in Figure 26. The central drains as well as lateral drains are usually made of cast iron, but these may also be made of other materials such as plastic, asbestos cement, concrete, etc.

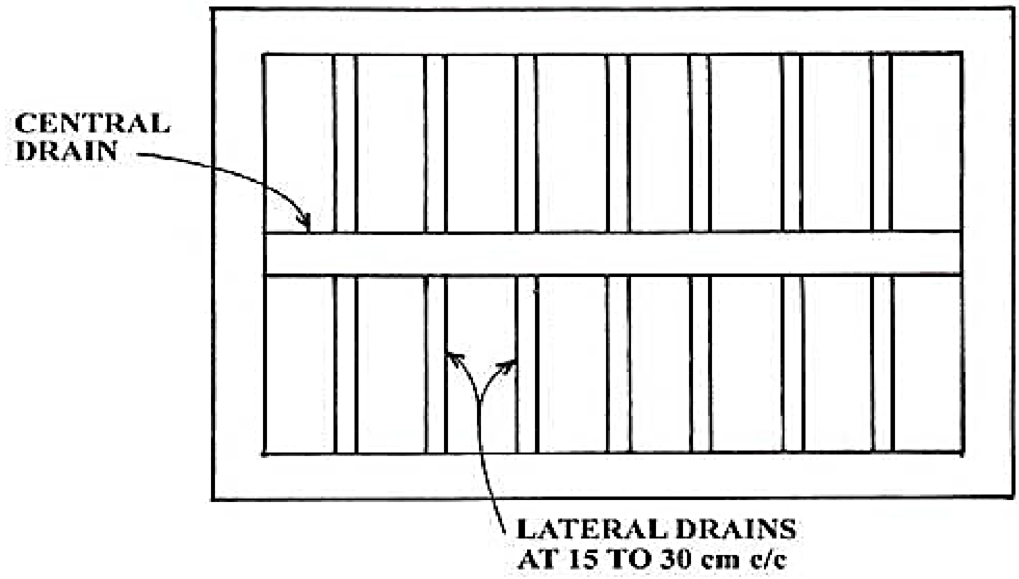


Figure 26: Underdrainage System for Rapid Sand Filter

The lateral drains are placed at a spacing of 15 to 30 cm. The lateral drains are provided with perforations on the bottom side which make an angle of 30° with the vertical as shown in Figure 27. The diameter of the perforations varies from 5 to 12 mm, and these are staggered. The spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm diameter to 200 mm for perforations of 12 mm diameter. The lateral drains are supported on concrete blocks of thickness about 4 to 5 cm placed on the floor of the filter.

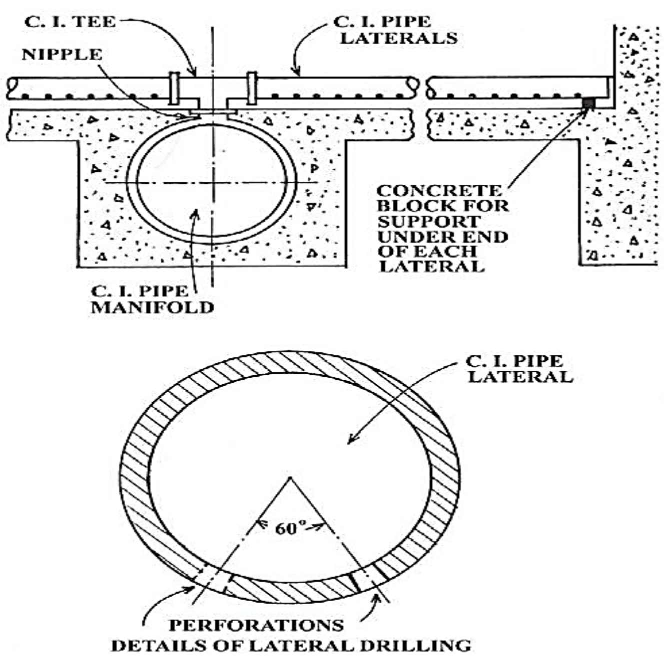


Figure 27: Perforated Pipe Underdrain

The perforated pipe system is economical and simple in operation. It, however, requires more quantity of water for back washing of filter, which is about 600 liters per minute per m^2 of filter area. The water for back washing is obtained either from a wash water overhead tank or by pumping. This is known as high velocity wash. The velocity of jet issuing from the perforations during back washing is, however, dissipated against the filter floor and in the surrounding gravel.

The following general rules may be observed in the design of an under drainage system consisting of central manifold and laterals.

(1) The ratio of length to diameter of the lateral should not exceed 60.

(2) The spacing of the laterals shall be from 15 to 30 cm.

(3) The cross-sectional area of the manifold should be preferably 1.5 to 2 times the sum of the cross-sectional areas of the laterals to minimize frictional losses and to give the best distribution.

(4) The diameter of perforations in the laterals should be between 5 and 12 mm. The perforations should be staggered, at a slight angle (usually 30°) with the vertical axis of the pipe.

(5) The spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm diameter to 200 mm for perforations of 12 mm diameter.

(6) The ratio of total area of perforations to the entire filter area may be about 0.003.

(7) The ratio of the total area of perforations in the under drainage system to the total cross-sectional area of the laterals should not exceed 0.5 for perforations of 12 mm diameter, and decrease to 0.25 for perforations of 5 mm diameter.

(b) Pipe and Strainer System

In this system, there is a central drain or manifold with lateral drains attached to it on either side. In this system holes are drilled at the top of the laterals and each hole is provided with a strainer as shown in Figure 28. A strainer is a small pipe of brass which is closed at top and contains holes on its surface. The strainers are either screwed or fixed on the top of the laterals drains. There are various forms of strainers devised by different manufactures of filter units. Some manufacturers provide umbrella shaped strainers.

Some umbrella shaped strainers have a special air orifice, and are employed where an auxiliary air wash is used. In some cases strainers are fixed even on the central drain. The lateral drains as well as the strainers are generally placed at a spacing of 15 to 30 cm. All the strainers are usually placed at the same elevation.

When pipe and strainer system is adopted, compressed air is used for the purpose of back washing of the filter. This results in saving of wash water. Thus it requires about 250 liters of water per minute per m^2 of filter area. This is known as low velocity wash.

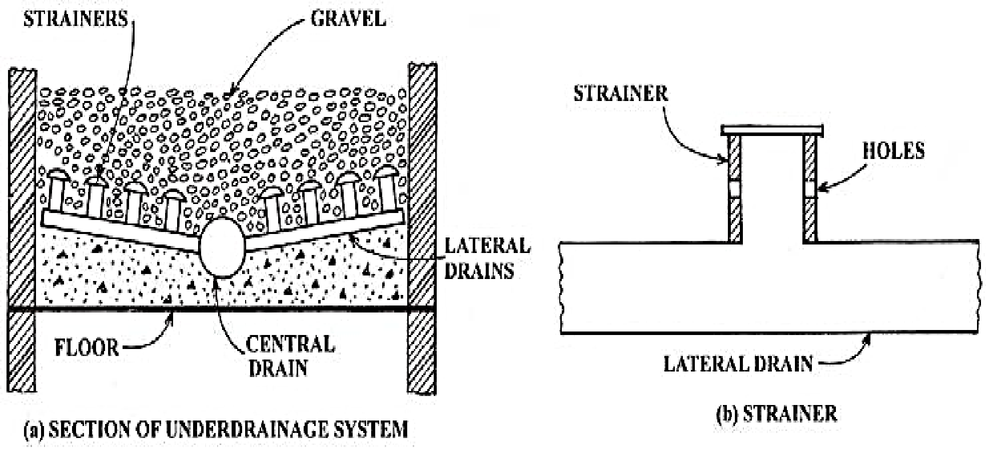


Figure 28: Pipe and Strainer Under drain

4.9.3.4 Appurtenances

The important appurtenances provided with a rapid sand filter are as follows.

(a) Wash Water Troughs

Wash water troughs are provided in the upper portion of the filter tank to collect the back wash water as it emerges from the sand and to conduct it to the wash water drain. These troughs may be of R.C.C., asbestos cement, plastic, cast iron and steel, out of which R.C.C. troughs are commonly used. The trough spans across the width or length of the tank. The spacing of the troughs is kept between 1.2 to 2 m, so that the horizontal distance traveled by the dirty water over the surface of the sand bed is kept between 0.6 to 1.0 m before entering the trough. The upper edge of the rough should be placed sufficiently near to the surface of the sand so that a large quantity of dirty water is not left in the filter after the completion of washing. At the same time, the top of the trough should be placed sufficiently high above the surface of the sand so that sand will not be washed into the trough. During back washing the sand is expanded to about 130 to 150 percent of its undisturbed volume, and hence the top edge of the trough should be slightly above the highest elevation of the sand as expanded in washing. Further the bottom of the trough should be kept at least 5 cm above the top surface of the expanded sand.

The troughs may be rectangular, square, V-shaped or semi-circular in section. The troughs having rectangular section at the top and V-shaped or semi-circular bottom are also used. The trough should be large enough to carry all the water delivered to it with a minimum freeboard of 5 cm. Any submergence of the trough will reduce the efficiency of the wash.

The troughs are designed as free falling weirs or spillways. For free falling rectangular trough with level bottom, the following expression is used for fixing the size of the trough.

$$Q = 1.376bh^{3/2}$$

Where

Q = total water received by the trough in m^3/s;

b = width of the trough in m; and

h = depth of water in the trough in m.

(b) Air Compressors

During back washing of a filter the agitation of the sand grains is carried out either by water jet, or by compressed air, or by mechanical rakes. When compressed air is used, an air compressor of required capacity should be installed. Generally it should have the capacity of supplying compressed air at the rate of 0.60 to 0.80 m^3 per minute per m^2 of filter area for 5 minutes. The pressure of the compressed air should be sufficient to overcome the frictional resistance offered by the air pipes and the depth of water lying above the air distribution system. The compressed air may be supplied either through the laterals of the under drainage system or through a separate pipe system. If the compressed air is supplied through the laterals of the under drainage system the pipe and strainer system of under drainage should be adopted. When a separate pipe system is to be provided for the supply of compressed air then the pipes and manifold of this system are placed immediately above the pipes of the under drainage system.

(c) Rate Control Device

It is essential to maintain a constant rate of filtration irrespective of loss of head through the filter. This is so because a sudden increase in the rate of filtration may cause water to break through the filter material without proper treatment, and sudden reduction in the rate of filtration may release a bubble of gas entrained in the sand, causing it to make a hole through the filter bed. In order to automatically control the rate of filtration the most commonly used device is *Simplex rate controller*, which is shown in Figure 29. It consists of a balanced valve connected to a flexible diaphragm (or disc) below, and to a lever with a movable weight above. The position of the valve controls the rate of flow through the device, and this position is regulated by means of the movable weight on the lever. The water enters this device through a venturi tube. A small pipe connection enables to transmit pressure at the throat of the venturi tube to the lower side of the diaphragm.

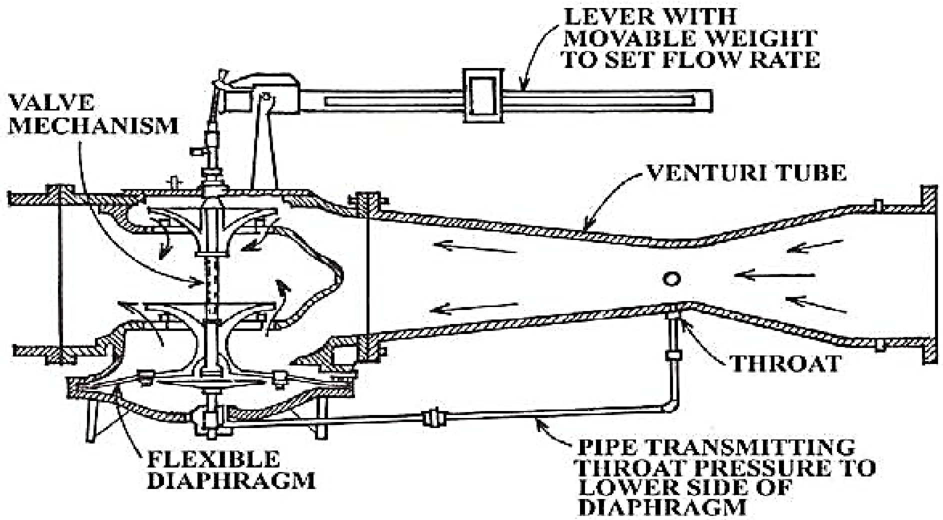


Figure 29: Simplex Rate Controller

By setting the movable weight on the lever, the position of the valve is so adjusted that the desired rate of flow is achieved. Corresponding to this rate of flow there will be certain pressure difference between the upper and the lower sides of the diaphragm which will balance the pull of the lever and the weight, and thus keep the valve in position to provide the desired rate of flow. When the rate of flow increases the pressure at the throat of the venturi tube will decrease, but in the control chamber where the valve is located the pressure will increase. Thus the upper side of the diaphragm will be subjected to a higher pressure and its lower side will be subjected to a lower pressure than before. The increased pressure difference between the upper and the lower sides of the diaphragm will force the valve to move down, thus reducing the rate of flow. Once the same rate of flow as before is restored the pressure difference between the upper and the lower sides of the diaphragm will be same as before and it will balance the pull of the lever and the weight and thus keep the valve in new position to provide the same rate of flow as before. When the filter gets clogged with its use, and the rate of flow decreases, the action in the rate controller is reversed and the same rate of flow as before is restored.

(d) Miscellaneous Accessories

The various accessories such as head loss indicators, meters for measuring the flow rates, etc., are also provided. The loss of head may be measured by inserting two piezometric tubes one above the sand bed in the filter tank, and other in the effluent pipe between the filter and the rate controller. Alternatively a differential manometer may be connected between these two points to measure the loss of head. Meters are installed for measuring discharges at inlet and outlet of the filter, and also at back wash.

4.9.3.5 Operation and Cleaning

The operation of a gravity type rapid filter shown schematically in Figure 30.

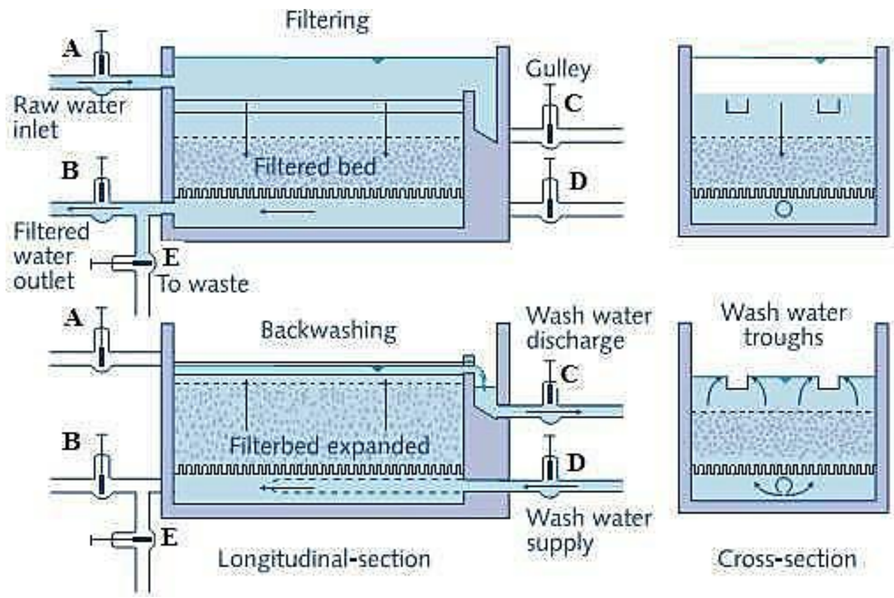


Figure 30: Operation of Rapid Sand Filter

During filtration the water enters the filter through valve A, moves down towards the filter bed, flows through the filter bed, passes the underdrainage system (filter bottom) and flows out through valve B. The unit used to measure filtration rate is actually the approach velocity, which is the inflow rate (m^3/h) divided by the filtration area (m^2). The interstitial velocity in the bed is higher, as it is the filtration rate divided by the average porosity of the filter medium.

Due to gradual clogging of the pores the filter bed's resistance against the downward water flow will progressively increase. This will reduce the filtration rate unless it is compensated by a rising raw water level above the filter bed. The rapid filters which have been designed to operate with a constant raw water level requires that the filter is equipped with a filter rate control device in the influent or effluent line. These filter rate controllers provide an adjustable resistance to the water flow. They open gradually and automatically to compensate for the filter bed's increasing resistance and so keep the operating conditions of the rapid filter constant.

When, after a period of operation, the filter rate controller is fully opened, further clogging of the filter bed cannot be further compensated and the filtration rate will fall. The filter is then taken out of service for cleaning. The cleaning of rapid sand filter is done by backwashing and surface washing of the filter.

4.9.3.6 Backwashing of Filter

The back washing of a filter is usually done when the loss of head through it has reached the maximum allowable value. The back washing of a rapid sand filter is carried out by passing water upwards through the filter bed. The following is the sequence of operations during back washing of the filter.

(i) Close valve A. Allow the filter to operate till the water level reaches the edge of the wash water troughs. Some operators permit the water level to fall about 15 cm from the top of the sand.

(ii) Close valve B.

(iii) Open valve D to allow wash water to pass through the filter bed in the upward direction. The valve should be opened gradually to prevent dislodging of the gravel. Open valve C to carry dirty wash water through inlet chamber to wash water drain. Continue washing till the wash water emerging from the filter bed and flowing through the wash water troughs appears to be fairly clear.

(iv) Close valve D.

(v) Close valve C after the water in the filter has drained down to the edge of the wash water troughs. Allow a short period to permit material in the water to settle on the surface of the sand and form a very thin sticky layer.

(vi) Open valve A slightly, and open valve E to allow the filtered water to flow to the wash water drain for a few minutes.

(vii) Close valve E and open valve B. Open valve A fully. The filter is now back in service.

Water used for back washing should be filtered water. The total wash water used should normally not exceed 2% of the treated water. The wash water should be applied at a pressure of about 5 m head of water as measured in the under drains. The rate of applications of wash water may be 600 liters per minute per m^2 of the filter surface area, equivalent to a rise in the filter tank of 60 cm per minute, for a period of 10 minutes. The capacity of the wash water storage tank should be sufficient to give a normal wash to two filters tanks for a period of 5 to 6 minutes.

Sometimes the backwashing of filter with water alone may not remove the dirt particles attached in the filter media. In such case, air wash system is employed in which compressed air is used to secure effective scrubbing action with a smaller volume of wash water. The air may be forced through the underdrains before the wash water is introduced or through a separate piping system placed between the gravel and the sand layer. Gravel is likely to be disturbed during air washing. The free air of about 0.60 to 0.80 m^3 per minute per m^2 of filter area at 35 kg/cm^2 is forced through the underdrains until the sand is thoroughly agitated, for a duration of about 5 minutes following which, wash water is introduced through the same underdrains at the rate of 400- 600 liters per minute per m^2 of the filter surface area.

The rapid sand filters get clogged very frequently and have to be back washed every 1 to 3 days. Normally 10 to 15 minutes are required in back washing, but in re-commissioning a total time of about 30 minutes may be consumed.

4.9.3.7 Surface Wash

The upper layer of the filter bed becomes the dirtiest because most of the impurities are trapped in this layer. This layer may not be properly washed by the conventional method of back washing. The inadequate washing of the top layer of the filter bed will lead to the formation of mud balls, cracks and clogged spots in the filters. These troubles may, however, be overcome by adequate washing of the upper layer of the filter bed by surface wash. The surface wash is usually accomplished by applying wash water or mechanical agitation, or both, at near the surface of the sand in the filter. The wash water is applied from above in the form of high pressure water jets directed on the surface of the sand bed. The two systems usually adopted for applying wash water on the surface of the sand are (a) fixed type surface wash system; and (b) rotary type surface wash system.

The fixed type surface wash system consists of pipes 25 mm or more in diameter arranged vertically at intervals of 0.6 to 0.9 m. The lower ends of the pipe are provided with nozzles and are kept about 0.1 m above the sand surface. An alternate fixed type surface wash system consists of pipes horizontally arranged at an interval of about 0.6 m at a height of 0.05 to 0.1 m above the sand surface. The horizontal pipes are perforated at intervals of about 0.3 m and provided with non-clogging orifices to prevent entry of filter media.

The rotary type surface wash system consists of rotating units suspended at a height of 50 to 75 mm at adequate intervals over the bed to provide complete coverage. Jet nozzles are located on side and bottom of arms and jet action of water causes the arms to rotate at a rate of 7 to 10 rpm.

The rate of application of surface wash water ranges from 200 to 4000 liters per minute per m^2 of filter area, under a pressure of 10 to 20 m head of water.

4.9.3.8 Filter Rate Control

Two types of filter rate control have been described below.

(a) Constant Rate Filtration by Influent Flow Splitting

In conventional rapid sand filters, constant rate of filtration is maintained by installing a rate of flow controller on the effluent line. These rate of flow controller can be quite complex and high in initial and maintenance costs. Alternative systems have been proposed which are relatively simple to build, operate and maintain.

One of the simplest methods is rate control by influent flow splitting which is depicted in Figure 31. The filter influent is divided equally among all the operating filters by means of a weir at each filter inlet. The size of the filter influent conduit is kept relatively large so that the head loss is not significant and the water level does not vary significantly along the length of the conduit. This helps in maintaining nearly same head on each of the weir and filter influent is equally split among all the operating filters. The filtration rate is controlled jointly for all the filter units by the inflow feeding rate. At the beginning of the filter run when a backwashed filter is put into service, the level of water in that filter is minimum. As the filtration proceeds and head loss builds up, the water level rises in the filter till it reaches the maximum permissible level above the filter bed, which may be, for example, equal to the level of influent weir. The filter is then taken out of service for backwashing.

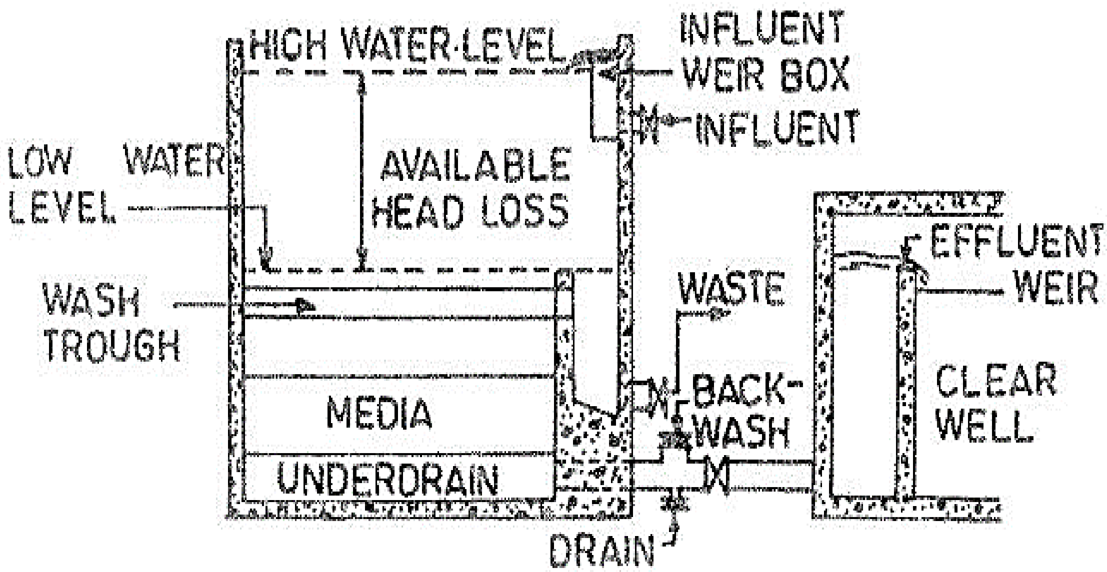


Figure 31: Rate Control by Influent Flow Splitting

The advantages of this system include elimination of rate controllers and smooth changes in rates due to gradual rise and fall of water level above filter bed with less harmful effects on filtrate quality in comparison to filters having rate of flow controllers.

To completely eliminate the possibility of negative head in the filter, the effluent control weir must be located above filter media.

The only disadvantages of the influent flow splitting system is the additional depth of the filter box which is 1.5 to 2 m more than in conventional filters.

(b) Declining Rate Filtration

This is also referred to as variable declining rate filtration. In this system, the filter influent enters below the water level of the filters and not above as in the case of influent flow splitting system. A relatively large influent header (pipe or channel) serve all the filters and a relatively large influent valve is used for each individual filter. This results in relatively small head losses in the influent header and influent valve and water level is essentially the same in all operating filters at all times. The essential features of variable declining rate of filtration system are shown in Figure 32. No rate of flow controllers are used in this system also.

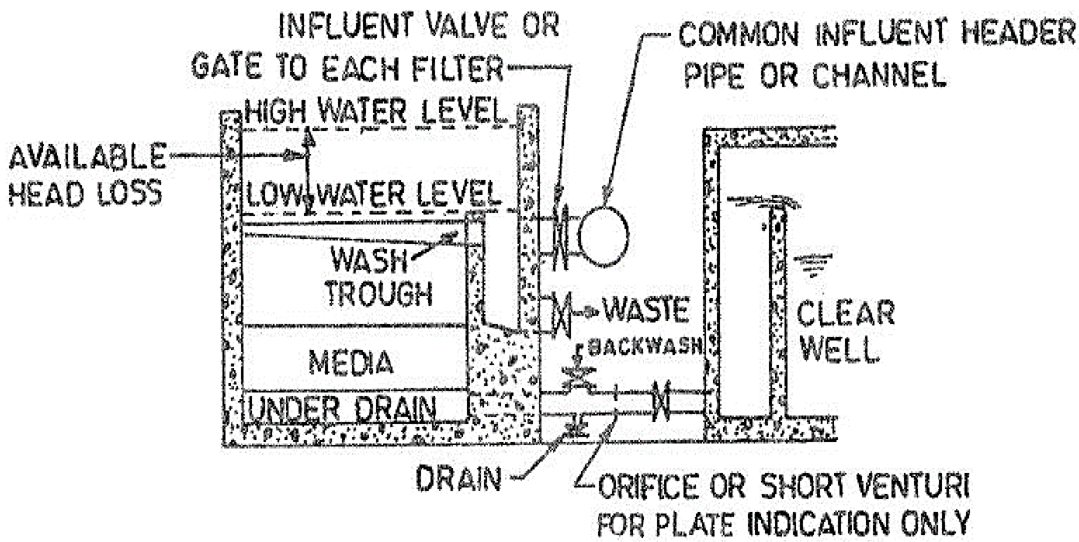


Figure 32: Variable Declining Rate of Filtration

During the course of filtration by a series of filters being served by a common header, as the filters get clogged, the flow through the dirtiest filters decreases most rapidly. This causes redistribution of load among all of the filters increasing the water level providing the additional head needed by the cleaner filters for handling additional flow. Therefore, the capacity lost by the dirtier filters is picked up by the cleaner filters.

The advantage claimed for this system include significantly better filtrate quality than obtained with constant rate filtration, and less available head loss needed than that required for constant rate operation.

4.9.4 Pressure Filters

Pressure filters are the types of rapid sand filters which are in closed steel cylindrical tanks and through which water to be treated passes under pressure. This pressure is developed by pumping head vary from 30 – 70 m. These filters may be of horizontal type or of vertical type as shown in Figure 33. The diameter of pressure filters vertical units varies from 0.4 m to 2.5 m and for horinzontal units varies from 2 m to 2.5 m varies from 1.5 m to 2.8 m and their length or height for vertical units varies from 2 m to 3 m and for horizontal units varies from 2.5 m to 7.5 m. At the top inspection windows (or manholes) are provided for the purpose of inspection. The wall metal thickness should be 8 to 10 mm and for cylindrical plant with 10 to 12 mm thickness for top and bottom depending on the pressure condition.

The metal is well protected inside with fiber reinforced plastic (FRP) lining and outside against corrosion.

The specifications for the sand and the gravel to be used for pressure filters are the same as those provided for rapid sand filters. Further the thickness of sand bed and gravel layers as well as the under drainage system are also the same as in the case of rapid sand filters.

The operation of pressure filters is similar to that of rapid sand filters except that in the case of pressure filters the coagulated raw water is fed directly to the filters without mixing, flocculation and sedimentation. The commonly used coagulant is alum, which is kept in a pressure container connected to the influent line to the filter.

The back washing of the pressure is also carried out in the same manner as in the case of rapid sand filters. Automatic pressure filters are also available in which back washing is done automatically after a fixed interval of time or when the head loss has reached a given value. It may, however, be noted that the head loss through pressure filters is approximately the same as through rapid sand filters. The term pressure filter does not imply that water is pumped through the filter under a high pressure loss.

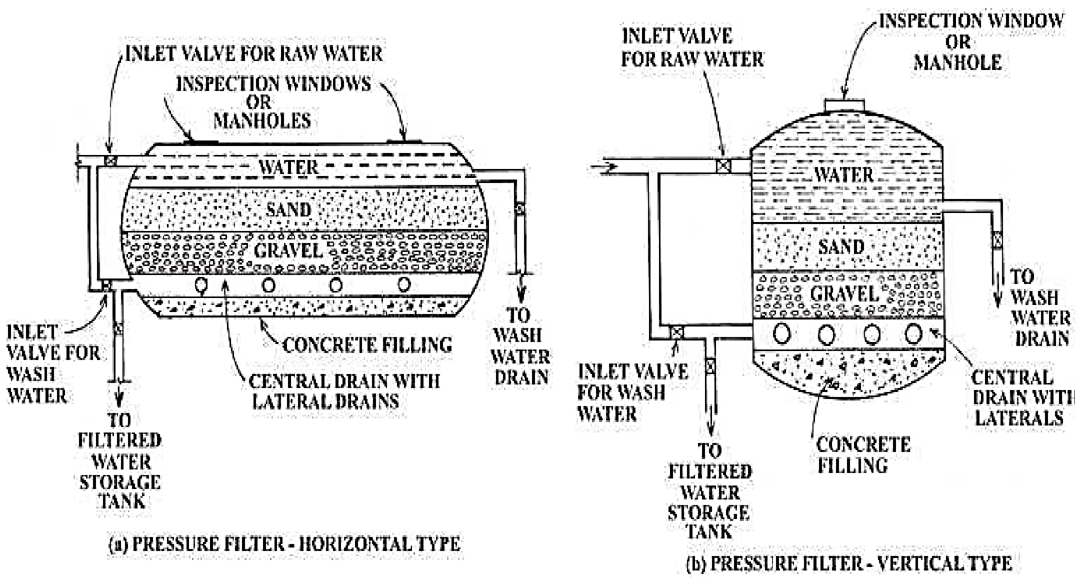


Figure 33: Pressure Filters

The air blower/compressor shall be able to supply air with air/water ratio of 10. It shall be able to supply air in sufficient quantity as specified at a pressure sufficient to move the air in the upward direction.

Rate of filtration: The rate of filtration of pressure filters is high as compared to that of rapid sand filters. It is about 120 -300 m/day of filter area.

Efficiency of pressure filters: In general pressure filters are found to be less efficient than rapid sand filters in terms of removal of bacterial load, color and turbidity.

Suitability of pressure filters: Pressure filters are not suitable for public water supply schemes, because of high cost, inefficiency of filtration, unreliable in removal of bacteria and relatively poor quality of results obtained. However, these filters can be installed for small water supply schemes such as colonies of a few houses, industrial plants, private estates, swimming pools, railway stations, etc and iron removal from ground water and in connection with softening.

4.10 Disinfection

The most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease or illness to the consumer. Processes such as storage, sedimentation, coagulation and flocculation, and filtration, both individually and jointly, reduce the bacterial content of water to varying degrees. However, these processes cannot assure that the water they produce is bacteriologically safe. Final disinfection will be needed for the complete removal of pathogenic organisms. Disinfection means the destruction, or at least the complete inactivation, of harmful micro-organisms present in the water. It is considered the last barrier in water treatment and in cases where no other methods of treatment are available, disinfection may be resorted to as a single treatment against bacterial contamination of drinking water.

The following factors influence the disinfection of water:

- The nature and number of the micro-organisms to be destroyed: Certain organisms like parasites and viruses may not be destroyed or completely inactivated by disinfection.
- The type and concentration of the disinfectant used: Higher concentrations are correlated to higher efficiencies.
- The temperature of the water to be disinfected: The higher the temperature the more rapid the disinfection will be.
- The time of contact: The disinfection effect becomes more complete when the disinfectant remains in contact with the water longer.
- The nature of water to be disinfected: If the water contains particulate matter, especially of a colloidal and organic nature (turbidity), the disinfection process generally is hampered due to the "protection" of the micro-organisms by the turbidity particles.
- The pH of the water: Chlorine, for example, will have better disinfection power if working at pH below 7, as the chlorine compound that will prevail is HOCl. At higher pH the chlorine compound present is OCl^-, which has a lesser bactericidal power.
- Mixing: Good mixing ensures proper dispersal of the disinfectant throughout the water, and so promotes the disinfection process.

In small communities there are two possible ways of disinfecting water for human consumption. If the population is scattered, disinfection can be applied at household level. In communities with a higher population density, a “central” water disinfection system is more efficient. As a classification of different disinfection methods, physical and chemical disinfection are discussed below.

4.10.1 Physical Disinfection

At family level the two principal physical disinfection methods used are boiling of the water and solar disinfection. Ultraviolet radiation has been gaining rising acceptance for community systems in developed countries, because of the reliability of the components and the declining costs.

4.10.1.1 Boiling

In some areas of the world this method may be expensive for the user (too much fuel consumption and work for women). Consumers usually do not like the taste of boiled water and it also takes a long time for the water to cool. Nevertheless, it is highly effective as a household treatment, as it destroys pathogenic micro-organisms such as viruses, bacteria, cercariae, cysts and ova. Boiling is normally carried out after education campaigns. In emergency situations, boiling of water may be used as a temporary measure. To enhance feasibility, promotion may focus on boiling water only for groups with the highest risks, such as infants and young children.

4.10.1.2 Solar Disinfection

Solar disinfection (SODIS) works on a different principle to that of boiling. SODIS uses pasteurization, which is based on the time/temperature relationship, to destroy pathogenic germs that may be present in the water. It has been observed that heating water above 62.8°C for 30 minutes or 71.7°C for 15 seconds is sufficient to remove waterborne bacteria, rotaviruses and enteroviruses from contaminated water. In addition, cysts of Giardia lambia are inactivated during 10 minutes at 56°C.

Popular SODIS is performed by beaming sunshine onto transparent water containers with exposure times of several hours. This technique is very appealing, as it does not depend on conventional energy, is very simple, and uses either bottles or low-cost equipment. It is environmentally friendly and people accept it without difficulty.

Nevertheless, SODIS has never reached peak popularity, as there are too many variables that influence the efficiency and eventual safety of the treated water. Parameters that may interfere with a perfect disinfection include geographical latitude and altitude; season;

number of hours of exposure; time of the day; clouds; temperature; type, volume and material of vessels containing the water; water turbidity and color.

The World Health Organization considers SODIS a valid option, but as a “lesser and experimental method”. Even so, for areas where there are no other means available to disinfect water, the method can substantially improve the bacteriological quality of water. The best results will be obtained when the measure is promoted and monitored by health officials or trained personnel (from a community-based or non-governmental organization – CBO or NGO).

Figure 34 shows both the batch and continuous solar disinfection systems. The continuous one comprises an exposure vessel or reactor and a tank where treated water exchanges heat with raw water (obviously without mixing).

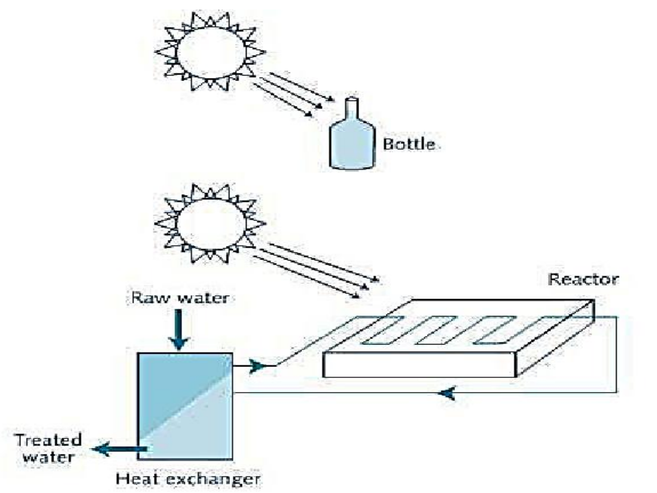


Figure 34: Batch and Continuous Solar Disinfection

4.10.2 Chemical Disinfection

Several chemicals, acting as strong oxidants, can destroy micro-organisms. Hydrogen peroxide and other metallic peroxides, lime, potassium and calcium permanganate, iodine, bromine, ozone and chlorine and its related compounds all fall into this category. Clean metals like copper, silver, mercury and zinc also disinfect, basing their action on a mechanism that is probably related to the absorption of the metallic ions by the organism, which in some way affects the chemistry of its cell structure. It is not only important to have the potential to destroy germs. A good chemical disinfectant for use in developing countries should also possess the following important characteristics:

- Quick and effective in killing pathogenic micro-organisms present in the water
- Readily soluble in water in concentrations required for the disinfection
- Capable of providing a residual
- Not imparting bad taste, odor or color to the water

- Not toxic to human and animal life
- Easy to detect and measure in water
- Not producing disinfection by-products (DBPs)
- Easy to handle, transport, apply and control
- Simple or “appropriate technology” devices for dosing
- Readily available
- Low cost

Unfortunately there is not one disinfectant that complies with all of those conditions. Almost all of them fall into a category that could be called “far from complying”. Only a few may be called “almost complying”.

4.10.3 Chlorination

Chlorination is the most widely used method of disinfection. It is both effective and economical, but must be administered constantly and safely. Chlorine is available in various forms, but all are corrosive and need careful handling. Chlorine gas is available in cylinders and drums and requires specialist equipment to enable the gas to be injected into a water flow. In powder form, chlorine is available as sodium hypochlorite (bleaching powder). A solution can be prepared from these powders and injected into the water flow at a rate proportional to the flow.

Even though chlorine and chlorine related substances are not perfect disinfectants, they have a number of characteristics that are highly valuable:

- They have a broad-spectrum germicidal potency and show a good persistence in water distribution systems. This means that they present residual properties that can be easily measured and monitored in networks or after the water has been treated and/or delivered to the users.
- Equipment needed for dosage is simple, reliable and low-cost. At village level, a number of “appropriate technology” feeders have proved to be easy to use, functional and accepted by local operators
- Chlorine or chlorine-based products are easily found even in remote locations in developing countries.
- It is very economic and cost effective.

In this method of disinfection chlorine (or its compounds) is used as disinfectant. Chlorine is an element, having the symbol Cl with an atomic weight of 35.45, melting point -101.5°C and boiling point 34.5°C. Gaseous chlorine is greenish yellow in color and is approximately 2.5 times heavier than air. Under pressure, it is a liquid with an amber color and oily nature approximately 15 times as heavy as water. Liquefaction of chlorine gas is accomplished by drying, cleaning and compressing the gas to 35 kg/cm^2. It is soluble in water to the extent of

4.61 volumes to 1 volume at 0°C and 2.26 volumes to 1 volume at 20°C, and the solution is called chlorine water. Chlorine water is unstable and its decomposition is particularly rapid on exposure to sunlight. Chlorine gas has a pungent odor which causes irritation when inhaled. It causes serious damage to lungs and other tissues even if present in the atmosphere in more than minutest traces and may result in death of the persons inhaling the gas. Severe coughing may be caused by the presence of 1 volume of the gas in 10000 volumes of air. Chlorine is non- combustible, but it supports combustion. In the presence of moisture, it is very active chemically and corrosive to metals. Active chlorine is the percentage by weight of molecular chlorine that would be rendered by a molecule of the compound. If, for example, a certain solution contains 10% of active chlorine, this is equivalent to 10 g of chlorine gas being bubbled (and totally absorbed) in 100 ml (100 g) of water.

4.10.3.1 Action of Chlorine

When chlorine is added to water, the following reaction takes place:

$$\mathbf{Cl2 + H2O \leftrightarrow HOCl + H^{+} + Cl^{-}} \text{ (Hydrolysis)}$$

This hydrolysis reaction is reversible. The hypochlorous acid (HOCl) dissociates I nto hydrogen ions (H^+) and hypochlorite ions (OCl^-) as indicated below.

$$\mathbf{HOCl \leftrightarrow H^{+} + OCl^{-}} \text{ (Ionization)}$$

This reaction is also reversible.

It is the hypochlorous acid (HOCl) and the hypochlorite ions (OCl^-) which accomplish disinfection of water. The un dissociated HOCl is about 80 to 100 times more powerful as disinfectant than OCl^- ion. Further the chlorine existing in water as hypochlorous acid, hypochlorite ions and molecular chlorine is defined as *free available chlorine.*

4.10.3.2 Chlorine Dose/Demand/Residual

The amount of chlorine added to the water is referred to as the dose, and is usually measured as the number of milligrams added to each liter of water (mg/l). The amount of chlorine destroyed in the reaction with the substances in the water is called the demand. The amount of chlorine (either free or combined) that remains after a certain contact time is known as the residual chlorine.

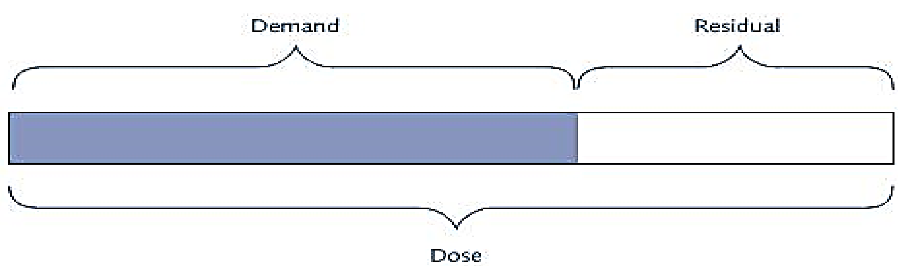


Figure 35: Relation between Dose/Demand/Residual Chlorine

Figure 35 illustrates that, if the dose is correctly applied, the water will take up the demand in order to achieve full disinfection, and there will still be a residual left that will cope with any post-contamination (for example in the distribution network). The residual is also important as a check on the effectiveness of the dosing, as monitoring the residual will test whether the disinfection treatment has been complete or not. When chlorine in the form of one of these compounds is added to water, a certain period of time is required for the chlorine to react with the micro-organisms and compounds in the water. This time is called the contact time, and a minimum of 30 minutes is usually recommended. The presence of the residual chlorine should be determined only after the specified retention time. If a 30 minutes retention time was set, then the monitoring should be done after that time has elapsed. This is what is called the C x T concept (concentration after a certain contact time).

WHO recommends the following conditions for a proper disinfection.

- Residual chlorine: ≥0.5 mg/l
- Contact time ≥30 minutes
- pH: < 8
- Turbidity: < 5 NTU; but ideally < 1 NTU

Several countries do not have, in their drinking water standards, an upper limit for residual chlorine in a distribution system. The WHO gives a guideline value of 5 mg/l. It is important that this value is not exceeded, as sometimes may happen in the first connections in the distribution network

4.10.3.3 Forms of Chlorine

Chlorine may be applied to water for disinfection in one of the following ways.

(a) As bleaching powder or hypochlorite
(b) As chloramines
(c) As chlorine gas or liquid chlorine

(a) Bleaching Powder

Bleaching powder or calcium hypochlorite Ca(OCl)2 is a chlorinated lime. When bleaching powder is added to water it dissociates into calcium Ca^{++} and hypochlorite OCl^- ions as indicated below.

$$\mathbf{Ca(OCl)2 \leftrightarrow \quad Ca^{++} + 2OCl^-}$$

The hypochlorite ions (OCl^-) combine with hydrogen ions present in water to form hypochlorous acid (HOCl) as indicated below.

$$H^{+} + OCl^{-} \leftrightarrow HOCl$$

This process is known as *hypo-chlorination*.

The bleaching powder contains about 30 to 35 percent of available chlorine. It is very unstable compound and it goes on losing its chlorine content when exposed to atmosphere, and hence it requires very careful storing.

The usual quantity of bleaching powder required for normal water is about 0.50 to 2.50 kg per million liters of water. The required quantity of bleaching powder is taken and dissolved in water to form a concentrated solution of it. This solution is the added in required quantity to water to be disinfected.

The process of hypo-chlorination is generally not adopted for large public water supply schemes. It may, however, be adopted for small installations such as small colonies, swimming pools, etc.

(b) Chloramines

Chloramines are the compounds formed by the reactions between ammonia and chlorine. These compounds are quite stable in water and remain in water as residuals for a sufficient time, contrary to the unstable chlorine which evaporates after some time. Since the disinfecting reactions are much slower in the case of chloramines, the water after treatment with chloramines should be supplied to consumers after an interval of about 20 minutes to 1 hour. Chloramines do not cause bad taste and odor when left as residuals, as is caused by chlorine alone. However, chloramines are much weaker disinfectants as compared to free chlorine.

(c) Chlorine Gas/Liquid Chlorine

There are two methods of application of chlorine to water to be disinfected:

(i) Chlorine gas may be fed directly to the point of application to the water supply, or

(ii) Chlorine gas may first be dissolved in a small flow of water and the chlorine water solution is fed to the point of application to the water supply.

The first method of application of chlorine is less expensive but it is less satisfactory because of poor diffusion of chlorine in water. Further it is found that at low temperatures (<10°C) crystalline hydrates of chlorine are formed, and hence when chlorine is directly fed through pipelines and if temperature falls down, choking of pipes may takes place. There is also a possibility of corrosion in pipes and valves resulting from accumulation of undissolved chlorine gas. As such only the second method of application of chlorine is used.

4.10.3.4 Pre and Post Chlorination

In a water facility, chlorination is normally performed at the end of the treatment, after the filtration stage. This is sometimes called post-chlorination. Pre-chlorination is sometimes applied prior to any other treatment. This is done for the purpose of controlling algae, taste and odor. In this case and when the raw water carries some organic materials (called precursors) it may give place to the production of disinfection by-products (DBPs). The most characteristic constituents of the DBPs are the trihalomethanes (THMs).

There has been some concern about THMs as some of them are carcinogenic. Though this is true, the risk of having widespread outbreaks of diarrheas and other water related diseases due to the lack of disinfection largely outweighs the risk of having some cases of cancer. The WHO and the USEPA strongly recommend not jeopardizing the microbiological safety of water in order to prevent eventual cases of cancer.

Volume 1 of the *WHO Guidelines for Drinking Water Quality* states: “An efficient disinfection should never be compromised”. Furthermore, the International Agency for Research on Cancer (IARC) in 1991 evaluated every available major scientific analysis of the potential health effects of chlorinated water, and concluded “chlorinated water is not a classifiable human carcinogen”.

4.10.3.5 Breakpoint Chlorination

When chlorine is added to water the following two actions take place:

(i) it kills bacteria present in water, thus disinfection is accomplished, and
(ii) it oxidizes the organic matter present in water, i.e., chlorine demand is satisfied.

The relationship between applied and residual chlorine is shown in Figure 36. Line A in the figure indicates the relationship of applied and residual chlorine in a pure water in which there is no chlorine demand and all the applied chlorine results into residual chlorine. . Line B in the figure indicates the relationship of applied and residual chlorine in a natural water which has chlorine demand. Since natural water may contain some ammonia, the initial reaction of the chlorine is with the ammonia to form chloramines, which are further oxidized by more chlorine to give a free chlorine residual. This is commonly called “breakpoint chlorination”. The break point in the chlorination of water is defined as the point on applied residual chlorine curve at which chlorine demand of water is satisfied after which all the applied chlorine appears as free residual chlorine.

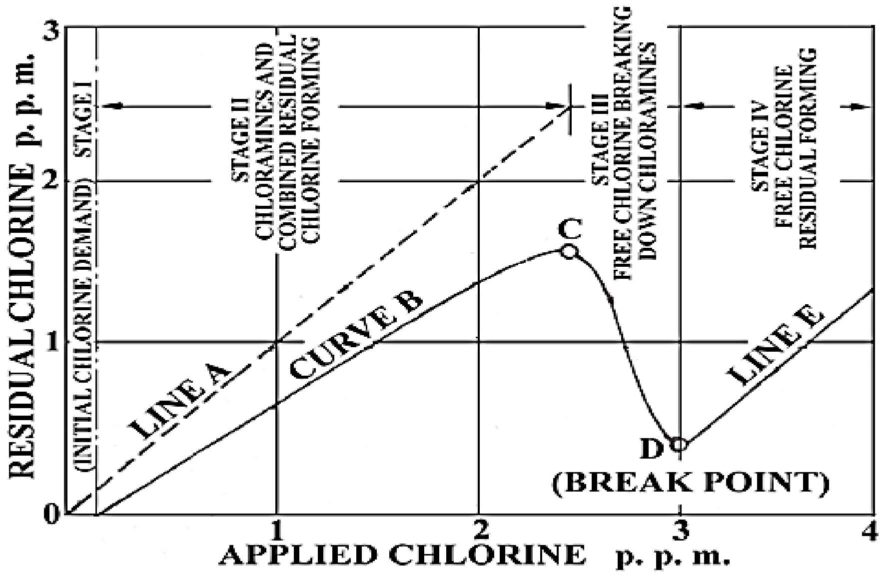


Figure 36: Breakpoint Chlorination

The application of chlorine to water should be done slightly higher than that at which break point occurs. Break point chlorination has the following advantages.

(i) It will remove taste and odor.
(ii) It will have adequate bactericidal effect.
(iii) It will leave desired chlorine residual.
(iv) It will complete the oxidation of ammonia and other compounds.
(v) It will remove manganese.

4.10.3.6 Application of Chlorine

Various methods of application of chlorine to water for disinfection are as follows:

(a) Chlorine Gas

Disinfection by gaseous chlorine is very economical and is the universally used technology around the world. More than 90% of the world population drinks water disinfected by chlorine gas. The most commonly used gas system consists of a cylinder with the gas, a regulator with a rotameter (feed rate indicator) and an injector as shown in Figure 37. The system operates under the vacuum created by water flowing past a venturi. A mixture of water and gas is injected at the application point, where the gas diffuses and dissolves.

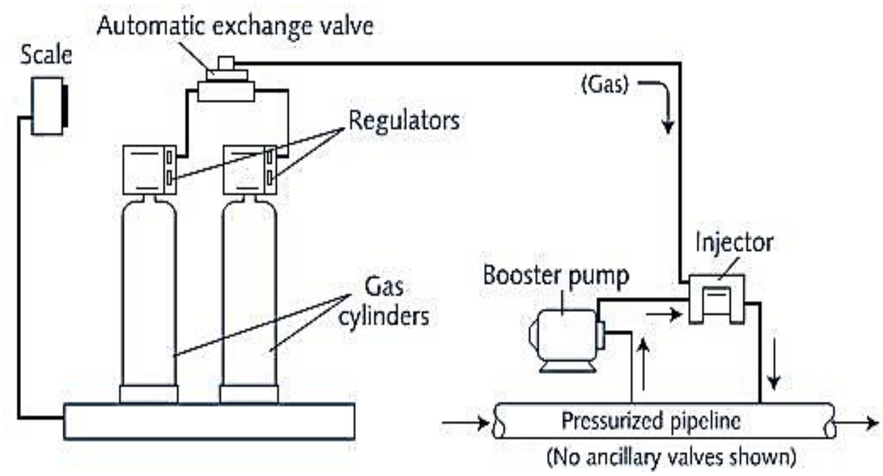


Figure 37: Chlorine Gas System

Even though the system is relatively simple, to ensure that the operation is safe and precise the personnel need proper training and several environmental safety precautions need to be in place (such as well-ventilated operating rooms, leak detectors, alarms, self-contained breathing apparatus, scales for gas cylinders, anchoring for the cylinders and safe gas transport systems). In general the costs of any chlorination method are quite low. However, in many countries the operating costs of chlorination using chlorine gas are about 25-50% of the cost of the equivalent solution of hypochlorite. But the capital investment costs required for gas chlorination, together with the precautions and training needs, make chlorine gas unfeasible for very small water supplies. Although chlorine gas systems are reliable, but for conditions prevailing in small communities, hypochlorite disinfection may be more reliable and simpler.

Pros	Widespread technology
	Chlorine gas is produced in almost every country
	Cheapest chemical
	Most widely used in the world
Cons	Costly system for very small villages
	Needs ancillary equipment
	Personnel need training
	Can be dangerous if not properly operated
	Not recommended for systems treating less than 500 m/day
O & M tips	Care should be taken on leaks
	Personnel should be strict on safety regulations and always use protective equipment

(b) Chlorine Solutions

All the other chlorine-based chemicals are liquid or can be dissolved and used as a solution. It is simple, easy, and low-cost appropriate technology devices can be used.

In the feeding of the chlorine solution, different dosing systems can be applied. These dosing systems can be subdivided into atmospheric pressure and positive pressure systems.

Under the heading of atmospheric pressure, the most popular methods are the ones using the constant head principle and the erosion system as shown in Figure 38.

Devices used include wheel feeders, suction feeders, or just feeding by hand (batch method usually used when a community tank is filled and then open for consumption).

A constant head system retains a stock solution at a fixed depth in a tank from which it is fed through a regulating valve to the water to be disinfected. An important precaution to be taken is to ensure that the stock chlorine solution does not have precipitates that may clog the valves. It is suggested always to install a small filter (comparable to the gasoline/diesel filters) upstream of the regulating valves.

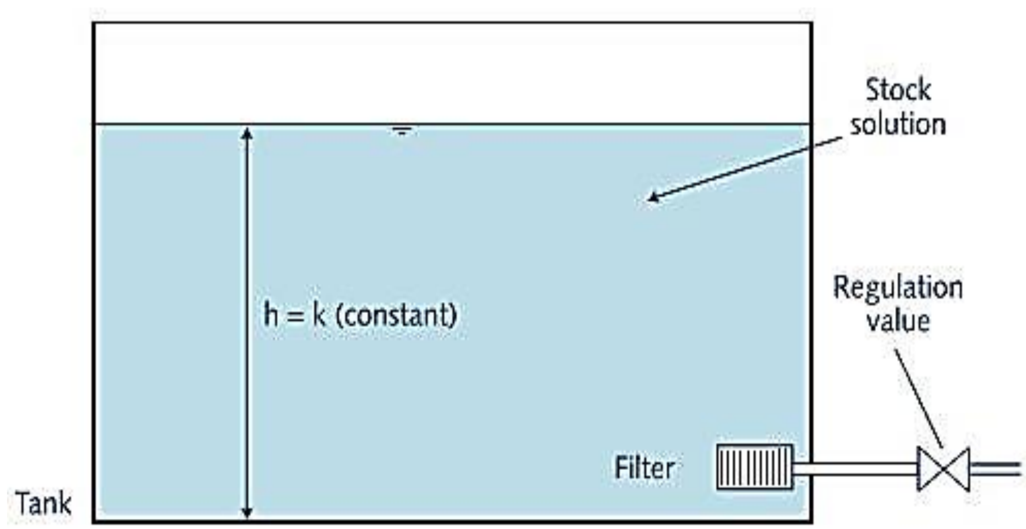


Figure 38: Constant Head System

These systems are used to dose chlorine solutions in channels or in tanks. There are a many such devices. Four of the most popular are shown here.

(1) Box Dosing System with Float Valve

The heart of this system is a float valve, the same kind as used in toilet cisterns as shown in Figure 39. One or two tanks hold the stock solution to be fed, and the float valve is placed in a small box. The system, although very simple, is cheap and accurate.

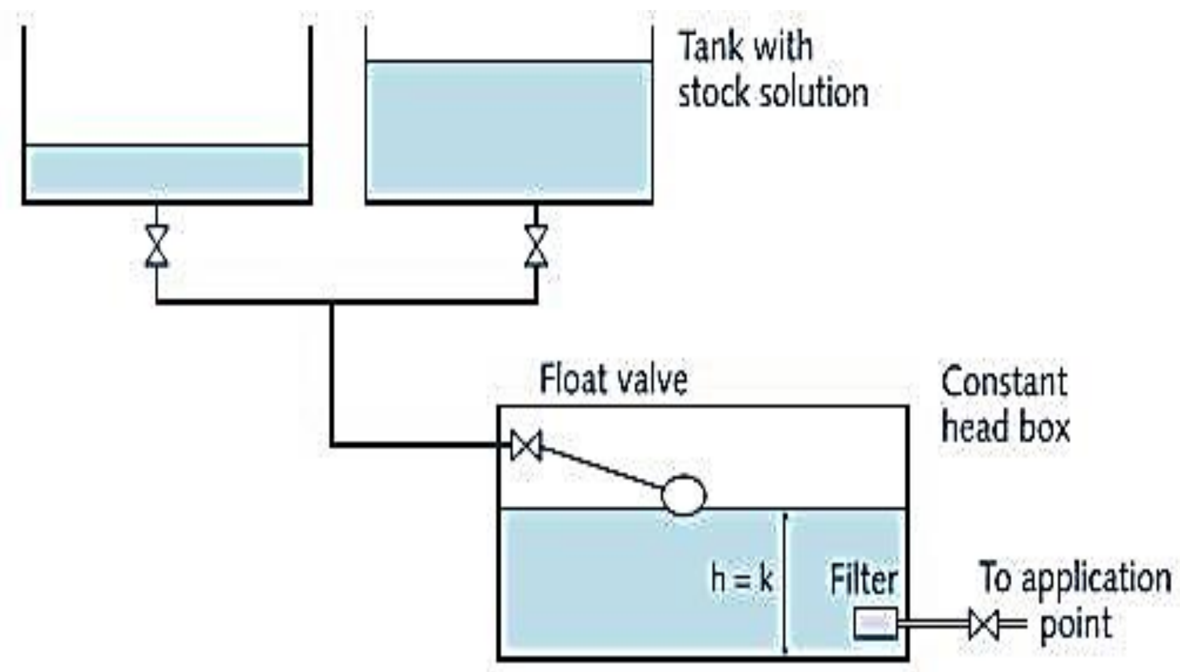


Figure 39: Box Dosing System with Float Valve

Pros	Extremely simple principle Very cheap Can be manufactured locally Reliable Does not need electric power
Cons	Error around 10% Material may corrode
O & M tips	Keep small orifices clean Use filter to eliminate particulate matter or sediments

(2) Hole Dosing System with Floating Tube

This, too, has been widely used in several different arrays. The basic element is a PVC tube with one or more holes. The tube is fixed to any kind of floating device and the hole/s should be placed some centimeters below the solution level. The tube is adjusted to such a level that the exact volume per second enters into the delivery tube and flows down to the application point. The floating tube with hole system is shown in Figure 40.

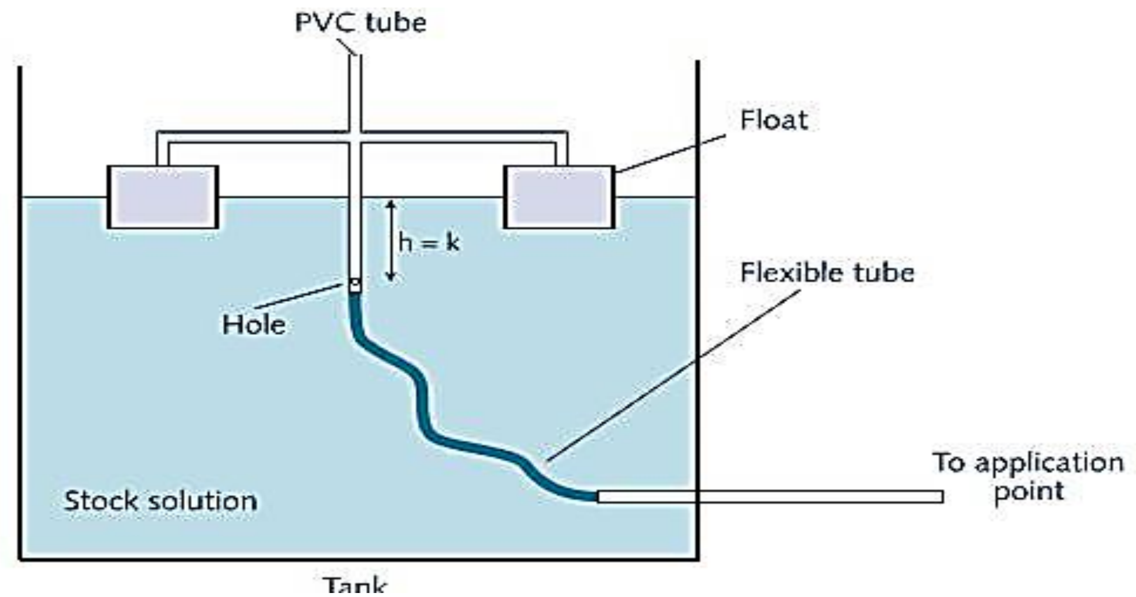


Figure 40: Hole Dosing System with Floating Tube

Pros	Extremely simple.
	Very cheap.
	Can be manufactured locally.
	Popular.
	Does not need electric power
Cons	Dosing error may be up to 20%
O & M tips	Keep small orifices clean.
	Use filter to eliminate particulate matter or sediments

(3) Bottle/Glass Dosing System

This system consists of a tank with the stock solution, a dosing element and a regulating valve as shown in Figure 41. The tank should be installed a meter or more above the level where the dosing element is placed. The dosing element is a simple system composed of a container with a floating device. It is made of a cylindrical plastic or glass bottle with smooth walls and with any volume between 0.5 and 1 liter. The bottle’s base should be removed and the bottle inverted (the neck facing down).

On the upper part (area of the removed base) a small cover made of wood or plastic is glued with epoxy putty. This cover has two holes. In the central one, a ¼ inch plastic tube or a piece of a discarded pen is introduced, protruding about 1 cm. This tube should be firmly welded or glued to the cover and its upper and lower edges should be levelled and smoothed. The second hole allows the air to flow freely.

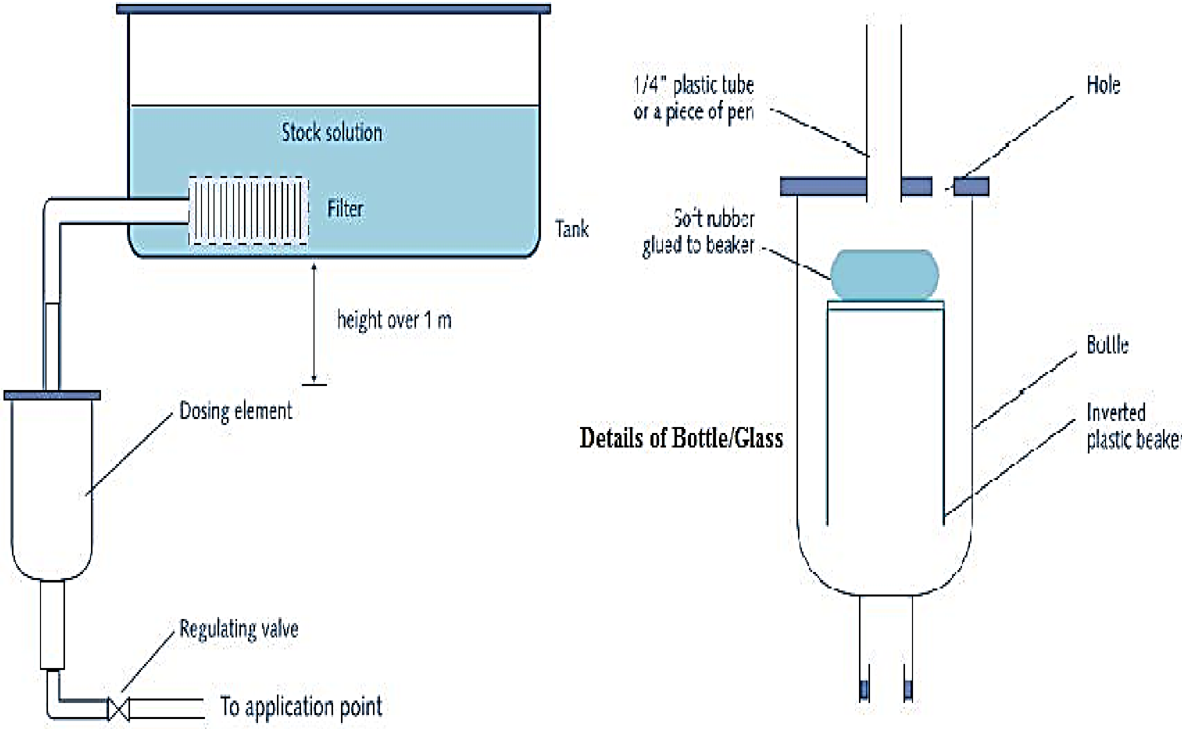


Figure 41: Bottle/Glass Dosing System

An inverted plastic beaker is placed inside the bottle. On the external part of the base, a piece of soft rubber is glued. Because air is trapped in the beaker, it operates as a floating device to regulate the flow to the bottle and the liquid level in the bottle. The flow to the water tank is regulated with a simple valve

Pros	Extremely simple Very cheap Can be manufactured locally Ideal for small communities Dosing error less than 10% Does not need electric power
Cons	Should be kept clean.
O & M tips	Use filter to eliminate particulate matter or sediments

(4) Diaphragm Pump Dosing System

Positive pressure feeders work on the principle that the chlorine solution is pressurized above atmospheric pressure and subsequently injected into a water pipeline. The most important positive pressure system is the highly popular diaphragm metering pump. These pumps are equipped with a chamber that has two one-way valves, one at the inlet and one at the outlet. The solution is drawn into the chamber through the inlet valve as the diaphragm opens, and is forced out of the chamber through the outlet valve as the diaphragm closes. An electric motor drives the diaphragm. The task of the pump is to elevate the solution by means of a series of strokes. The application point may be a channel or reservoir (atmospheric pressure), but also a pipeline with running water (positive pressure). The diaphragm pump dosing system is shown in Figure 42.

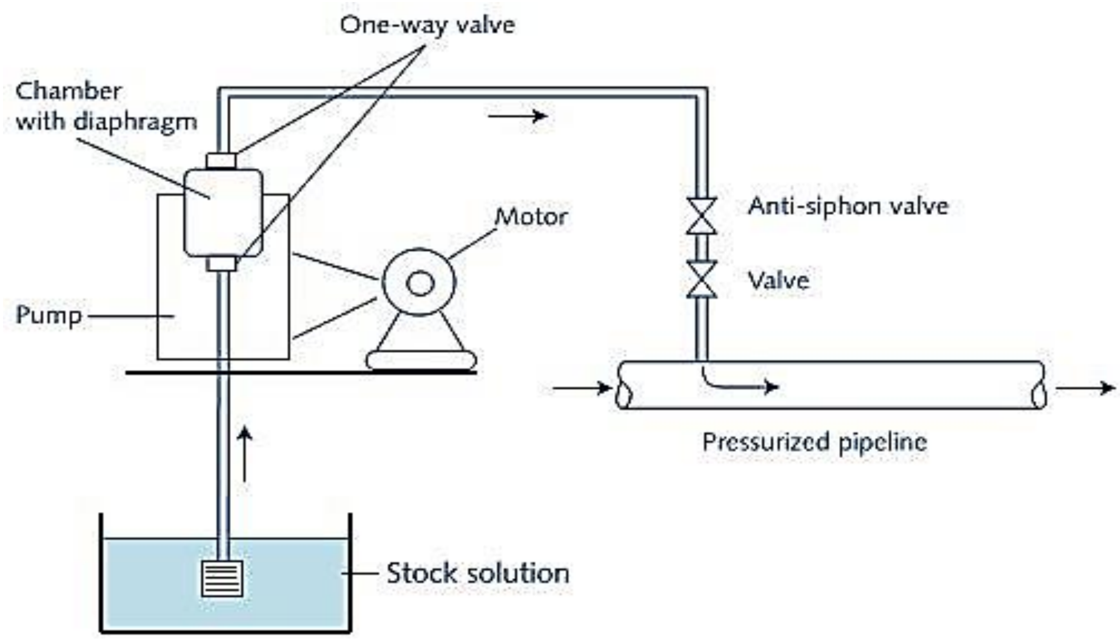


Figure 42: Diaphragm Pump Dosing System

Pros	Highly reliable
	Very popular
	Simple to operate
	One of the few systems to work under pressure
Cons	Personnel should be trained in its operation and maintenance
	Intermediate to high cost for a rural system
	Needs electric power
O & M tips	Give continuous and proper maintenance
	Check the anti-siphon valve

A second way to inject a chlorine solution in a pressurized pipeline is shown in Figure 43. This system is very simple and economical, but if the mixing of the injected stock solution in the pipeline is not good, it may cause damage to the pump turbine.

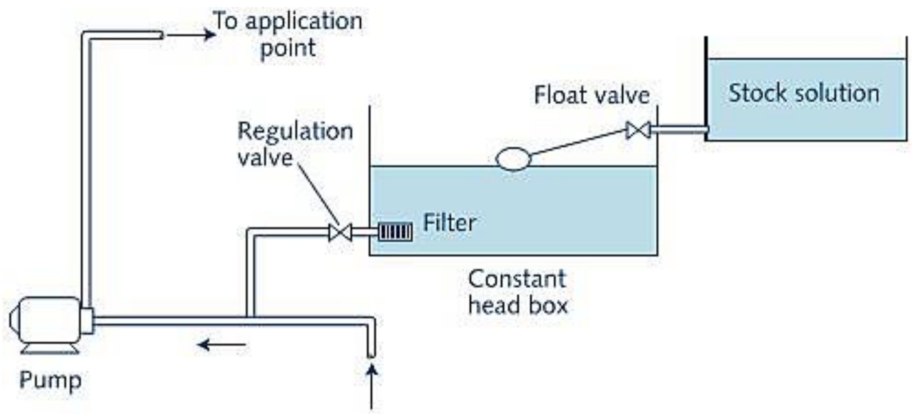


Figure 43: Chlorination in Pressurized Pipeline

Pros	Very simple
	The cheapest solution for a feed under pressure
Cons	It should be monitored
	Sometimes there is corrosion in the pump rotor due to the chlorine
O & M tips	Use filter to eliminate particulate matter or sediments

4.11 Softening

4.11.1 General

A water is said to be hard when it does not foam lather readily with soap. The hardness of water is due to the presence of calcium and magnesium ions in most cases. Bicarbonates, sulphates and chlorides are the anions associated with the hardness. The purpose of softening is to remove these salts from the hard water, to ensure longer life to washed fabrics, mitigate its scale forming tendencies and improve palatability.

According to the NDWQS the total hardness in drinking water should not be more than 500 mg/l as $CaCO_3$. The magnesium hardness in general should not exceed 40 mg/l to minimize the possibility of magnesium hydroxide scale in domestic hot water heaters and pipelines. Calcium and magnesium associated with bicarbonates are responsible for carbonate hardness and that with the sulphates, chlorides and nitrates contribute to non-carbonate hardness.

Normally, the alkalinity measures the carbonate hardness unless it contains sodium alkalinity. The non-carbonate hardness is measured by the difference between the total hardness and the carbonate hardness. Carbonates and bicarbonates of sodium are described as negative non-carbonate hardness.

The two methods ordinarily used are lime and lime soda softening and ion-exchange softening.

4.11.2 Lime and Lime Soda Softening

Softening with lime and soda is used particularly for water with high initial hardness (greater than 500 mg/l) and suitable for waters containing turbidity, color and iron salts because these have tendency to inactivate the ion-exchange bed by a coating on the granules. Lime soda softening cannot, however, reduce the hardness to values less than 40 mg/l while ion-exchange softening can produce a zero hard water.

When lime and soda are added to water containing calcium and magnesium salts, the following reaction takes place.

$CaO + H_2O \rightarrow Ca(OH)_2$ (Quicklime slaking)

$Ca(OH)2 \leftrightarrow Ca^{++} + 2OH^-$

$CO2 + OH \leftrightarrow HCO3$ (Removal of CO2)

$HCO3^- + OH^- \leftrightarrow CO3^- + H2O$

$Ca^{++} + CO\text{-}_3 \leftrightarrow CaCO_3$(Removal of calcium hardness)

$Mg^{++} + 2OH^- \leftrightarrow Mg(OH)2$ (Removal of magnesium hardness)

On a 100% purity basis, the dosage of lime as CaO required for softening as obtained from the chemical equations is presented in Table 14.

Table 14: Amount of CaO for Softening

S.No.	Particulars	CaO Required	
1	For every gram of CO_2 to be removed	1.27	gm
2	For every gram of CO_3 hardness as $CaCO_3$ to be removed	0.56	gm
3	For every gram of Mg as Mg to be removed	2.33	gm
4	Additional lime required for raising the pH to the range of 10 to 10.5 for precipitation of $Mg(OH)_2$	30-50	mg/l
5	Soda ash requirements as Na_2CO_3 to remove one gram of non-carbonate hardness as $CaCO_3$	1.06	gm
6	Additional soda required to neutralize every gram of excess lime	1.89	gm

Alternatively, caustic soda can be used instead of lime. The amount of calcium carbonate sludge formed in this case is theoretically half that formed by use of lime. However, caustic soda is costlier than soda ash which is more expensive than lime.

Waters higher in carbonate than in non-carbonate hardness will require relatively more lime than soda ash for their treatment. The sludge settling out carries with it a large portion of the turbidity, iron, manganese, silica, color producing matter bacteria in the water. The softened water may have a higher pH due to neutralization of CO2 with lime, thus reducing its corrosive nature.

Lime soda softening plants include chemical feeders, rapid mix, flocculation and sedimentation basins and rapid sand filters. Figure 44 shows the complete treatment process of lime soda softening. The process design must ensure the promotion of the chemical reactions necessary to remove the hardness from water by converting them into insoluble precipitates and then settle these precipitates and filter the partially clarified water.

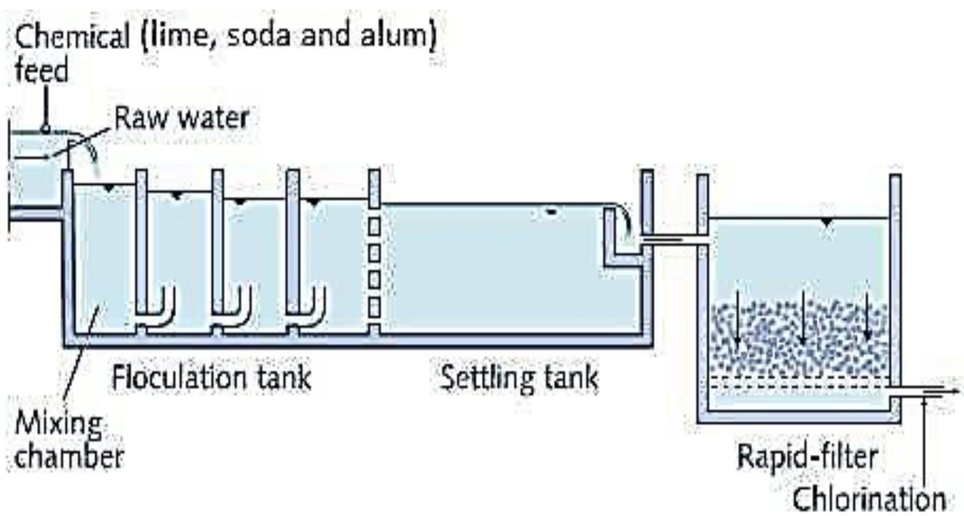


Figure 44: Treatment Processes in Lime Soda Softening

The solubility of calcium hydroxide (slaked lime) is low being of the order of 1600 to 1800 mg/l in cold water. It is commonly practice, therefore, to feed a 5% slurry of lime so that unusually large solution tanks are avoided. Soda ash is added in solution form.

4.11.3 Ion Exchange Softening

The ion exchange process is the reversible inter-change of ions between a solid ion exchange medium and a solution and is used extensively in water softening. The hardness producing ions preferentially replace the cations in the exchangers and hence this process is also known as Base Exchange softening. The ion exchange can produce a water of zero hardness. There is only a temporary change in the structure of the exchange material which can be restored by regeneration. The ion exchanger can work on the hydrogen or sodium cycle, the hydrogen ion being released into the water in the former case and the sodium ions in the latter. The regenerants are an acid and sodium chloride respectively. In general the ion exchange materials used in softening, also called zeolites, are hydrated silicates of sodium and aluminum having the formula xNa2O. yAl2O3. Z SiO3. The reaction can be depicted as follows.

$$Na_2Z + \frac{Ca^{++}}{Mg^{++}} + 2A^- \leftrightarrow \frac{Ca}{Mg}Z + 2Na^+ + 2A$$

Where A represents the relevant anions of bicarbonates, sulphates or chlorides and Z represents anionic part of the zeolite.

Treatment with zeolite thus increases the dissolved solids in the ratio of 46:40 of the hardness removed. The reverse equation operates during the regeneration resulting in a strong solution of calcium and magnesium salts, which is run to waste.

(a) Inorganic Zeolites

The two common inorganic zeolites are the natural and synthetic types. The natural zeolite is available as green sand while the synthetic or gel type is obtained by the reaction of either sodium aluminate or aluminum sulphate with sodium silicate which after drying is graded to suitable sizes by screening.

(b) Organic Zeolites

The organic zeolites consist of sulphonated carbonaceous material and sulphonated styrene type resins which have excellent cation exchange properties, requiring foe regeneration 2-4 kg of salt for every kg of hardness removed. These are resistant to attack by acid solutions and hence can be used for waters with a wide pH range. The loss due to attrition is negligible compared to synthetic inorganic zeolites. They are lighter than inorganic zeolites.

(c) Raw Water Characteristics

For application to the ion exchangers, the raw water should be relatively free from turbidity, as otherwise the exchange material gets a coating which affects the exchange capacity of the bed. The desirability of using filters prior to zeolite bed or restoring to more frequent regeneration would depend upon the level of turbidity. Metal ions like iron and manganese, if present are likely to be oxidized and can coat the zeolites, thus deteriorating the exchange capacity steadily since the regenerant cannot remove these coatings. Oxidizing chemicals like chlorine and carbon dioxide as well as low pH in the water will have a tendency to attack the exchange material particularly the inorganic types, the effect being more pronounced on the synthetic inorganic zeolites. The organic zeolites operating on brine regeneration cycle do not add any silica to the water and consequently are ideally suited for boiler feed water.

(d) Design Criteria

The design criteria foe ion exchange softening system is based upon

(i) the required flow rate,
(ii) the influent water quality,
(iii) desired effluent water quality
(iv) exchange capacity and hydraulic characteristics of the exchanger,
(v) period between regenerations,
(vi) type of operation,
(vii) number of units required,
(viii) rate, time of contact, uniformity and concentration of brine application,
(ix) rate and volume of rinse and
(x) Quality of regenerant.

A softening unit is similar to a rapid sand filter unit regarding the hydraulics and equipment.

Volume of exchange material to be used in cubic meters (E) is calculated by the formula.

$$\boldsymbol{E = \frac{QH}{1000G}}$$

Where,

Q = Volume of water to be treated between regenerations, in m^3

H = Hardness of water in mg/l

G = Exchange capacity of the material in kg/m^3

Generally, ion exchange beds are encased in shells, shell diameter and bed depth being adjusted to maintain a rinse rate of flow in the range of 200 to 400 m/day. The vertical units are 0.2 to 3.0 m in diameter while the horizontal ones are 3 m in diameter and 6 - 7 m long.

The ion exchange bed has a depth of 0.6 m usually and is placed over supporting gravel (size depending upon composition of the exchange material but with similar specification as those for rapid gravity sand filters) of 0.30 to 0.45 m depth with an underdrain system at the bottom foe collecting softened water. After the softening cycle, the softener should be backwashed for 3 to 5 minutes to loosen the exchange resin and remove particulate matter.

The rate of backwash should ensure at least 50% bed expansion. The regeneration of the bed is carried out with brine solution. The brine distribution manifold is placed immediately above the softener bed.

Exchange capacities and the common salt requirements of cation exchange are presented in Table 15.

The optimum concentration of brine for restoration of maximum exchange capacity in any resin is about 10 to 15% and the contact time for regeneration varies from 20 to 45 minutes. A dosage of salt of 15 kg/min m^3 of resin using 10% brine solution is usually applied at the rate of about 150 lpm/m^3 of exchanger.

Table 15: Exchange Capacity of Cation Exchangers

Cation Exchanger	Capacity (kg/m^3)	Common Salt kg/kg exchanged
Natural		
Green sand	7-14	3.5-7.0
Synthetic inorganic		
Synthetic siliceous zeolite	14-37	2.5-3.5
Synthetic organic		
Sulphonated coal	12-70	2.0-4.0
Resin polystyrene	25-100	2.0-4.0

The total rinse water requirement is 3 to 10 m^3/m^3 of material and applied at a rate of 9 to 18 $m^3/h/m^3$ in the slow and 30 $m^3/h/m^3$ in the fast types. The rinse water is introduced through the brine distribution network or by simply flooding the unit through a hose.

The salt or brine storage tank should provide for a capacity of 24 hours or 3 successive regenerations, whichever is greater.

4.12 Removal of Iron and Manganese

All the water whether it is surface or ground contains some traces of iron. It is because iron is practically present in all types of soils and rocks and the rain water percolating through these soils and rocks acquires iron and other minerals. Further manganese may accompany iron in water in smaller amounts.

Iron and manganese are generally present in water either in suspension as hydrated oxides or in solution as bicarbonates. The iron present in water in suspension as hydrated oxides can be removed by the normal treatment methods of coagulation, sedimentation and filtration. The iron present in water in solution as bicarbonates requires special treatment methods. The iron and manganese should not be more than 0.3 and 0.2 mg/l respectively in drinking water as per the National Drinking Water Quality Standards. The amount of iron and manganese higher than the value specified in the standards should be removed by treatment. The iron and manganese have following objectionable effects.

(i) It produces unpleasant taste and odor in water.
(ii) It causes staining of plumbing fixtures, clothing and textiles.
(iii) Iron and manganese may deposit in the pipes leading to its blockage.
(iv) The water becomes red or brown in color.
(v) There may be growth of iron bacteria in water mains.
(vi) It causes troubles in various manufacturing processes.
(vii) It causes corrosion of plumbing works.

Iron and manganese present in water can be removed by the following methods.

(a) Aeration followed by Sedimentation and Filtration

When iron and manganese are present in water without organic matter it can be removed by aeration followed by coagulation, sedimentation and filtration. During aeration dissolved ferrous and manganese compounds are converted into insoluble ferric and manganese compounds which can then be removed in settling tank or filters or both.

The following reaction takes place during the aeration in case of iron.

$$\mathbf{4Fe(HCO_3)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \downarrow + 8CO_2}$$

Above equation shows that soluble ferrous bicarbonate is converted into insoluble ferric hydroxide. A reaction time of about 5 minutes or less is required at pH of 7 to 7.5 and 0.14 mg of oxygen is required to precipitate 1 mg of ferrous bicarbonate to ferric hydroxide. Hence, slight agitation of water with air is usually sufficient.

In case of manganese the following reaction takes place.

$$\mathbf{6Mn(HCO_3)_2 + 3O_2 \rightarrow 6MnO_2 \downarrow + 2CO_2 + 2H_2O}$$

As per the above equation, 0.29 mg of oxygen is required for 1 mg of manganese.

Manganese removal requires a pH adjustment up to 9.4 to 9.6.

In order to accelerate the oxidation particularly in waters with high carbon dioxide, addition of alkaline substances such as lime, soda ash or caustic soda is done which enhances the iron and manganese removal by catalytic action.

When iron and manganese occur in water in combination with organic matter, it becomes difficult to break the bond between them. Iron and manganese will not be converted to insoluble form until the bond between them is broken. The bond can be broken either by adding the lime raising the pH value of water to about 9 or by adding chlorine or potassium permanganate. Once the bond is broken, iron and manganese can be removed by the treatment as mentioned above.

(b) Base Exchange Method

When water does not contain large amounts of iron and manganese, these can be removed by means of manganese zeolite. As raw water passes through the bed of zeolite, the iron and manganese ions substitute the sodium ion present in the zeolite thus by removing iron and manganese from water. The manganese zeolite also oxidizes the iron and manganese to insoluble hydrated oxides which are removed by the filtering action of the zeolite bed. The bed must be washed out and regenerated occasionally with potassium permanganate.

(c) Chlorination followed by Sedimentation and Filtration

Iron and manganese can also be removed by their oxidation using chlorine and then followed by sedimentation and filtration. The oxidation of iron, particularly of organic origin, can be achieved by the use of potassium permanganate.

4.13 Lime Scale Deposit Removal and Prevention

4.13.1 General

Lime scale is the hard, off-white, chalky deposit that builds up due to calcium carbonate deposits in water. It is normally found in kettles, hot-water boilers and the inside of inadequately maintained hot-water central heating systems. It is also often found as a similar deposit on the inner surface of pipes and other surfaces where "hard water" has evaporated. In addition to being unsightly and hard to clean, lime scale seriously impairs the operation or damages various components including reduced water flow in taps and pipes, build-up on bathtubs, taps and sinks, and preventing heat transfer in kettles due to insulation. If lime scale deposits are not removed, it will ultimately clog the pipeline and plumbing fixtures and consume more energy for heating and boiling of water.

4.13.2 Equilibrium Condition

Hard water contains calcium and often magnesium bicarbonate or similar ions. Calcium salts, such as calcium bicarbonate and calcium carbonate are both more soluble in hot water than cold water. Thus, heating water does not cause calcium carbonate to precipitate. However, there is equilibrium between dissolved calcium bicarbonate and dissolved calcium carbonate as shown by the following equation:

$$\mathbf{Ca^{2+} + 2HCO3^{-}\ Ca^{2+} + CO3^{2-} + CO2 + H2O}$$

Where the equilibrium is driven by the carbonate/bicarbonate, not the calcium. Note that the CO2 is dissolved in the water.

There is also an equilibrium of carbon dioxide between dissolved in water (dis) and the gaseous state (g): $CO_2(dis) \leftrightharpoons CO_2(g)$

The equilibrium of CO2 also moves to the right towards gaseous CO2 when the water temperature rises. When water that contains dissolved calcium carbonate is heated past 55°C or left to stand, CO2 is removed from the water as gas causing the equilibrium of bicarbonate and carbonate to shift to the right, increasing the concentration of dissolved carbonate. As the concentration of carbonate increases, calcium carbonate precipitates as the salt: $Ca^{2+} + CO_3 \leftrightharpoons CaCO_3$. As new cold water with dissolved calcium carbonate/ bicarbonate is added and heated, CO2 gas is removed, carbonate concentration increases, and more calcium carbonate precipitates.

Such a phenomenon also occurs during the aeration of water where CO2 gas is released thus precipitating the calcium carbonates in pipelines and plumbing fixtures.

4.13.3 Langelier Saturation Index

The Langelier Saturation Index (LI), a measure of a solution's ability to dissolve or deposit calcium carbonate, is often used as an indicator of the corrosivity of water. The index is not related directly to corrosion, but is related to the deposition of a calcium carbonate film or scale; this covering can insulate pipes, boilers and other components of a system from contact with water. When no protective scale is formed, water is considered to be aggressive and corrosion can occur. Highly corrosive water can cause system failures or result in health problems because of dissolved lead and other heavy metals. An excess of scale can also damage water systems, necessitating repair or replacement.

In developing the LI, Langelier derived an equation for the pH at which water is saturated with calcium carbonate (pHs). This equation is based on the equilibrium expressions for calcium carbonate solubility and bicarbonate dissociation. To approximate actual conditions more closely, pHs calculations were modified to include the effects of temperature and ionic strength.

The Langelier Index is defined as the difference between actual pH (measured) and calculated pHs and is represented by the following equation.

$$\mathbf{LI = pH_{actual} - pH_s}$$

Where, $\mathbf{pH_s = A+B-C-D}$

Constant A takes into account the effect of temperature. It is found by selecting the value from the *Water temperature* table that corresponds to the measured temperature in degrees Celsius.

Constant B is a correction for the ionic strength of the sample. It is determined using the *TDS* table by taking the value that corresponds to the measured total filterable residue or the estimated total dissolved solids (TDS).

Value C is obtained from the Hardness *or alkalinity* table by reading the value corresponding to the calcium hardness (in mg/L CaCO3) of the sample.

Value D is obtained from the Hardness or alkalinity table by reading the measured value for total alkalinity (in mg/L CaCO3) of the sample.

The values of A, B, C and D are presented in Tables 16, 17 and 18.

The LI is a gauge of whether water will precipitate or dissolve calcium carbonate. If the pHs is equal to the actual pH, the water is considered “balanced”. This means that calcium carbonate will not be dissolved or precipitated. If the pHs is less than the actual pH (the LI is a positive number), the water will tend to deposit calcium carbonate and is scale-forming (nonaggressive). If the pHs is greater than the actual pH (the LI is a negative number), the water is not saturated and will dissolve calcium carbonate (aggressive). In summary:

$pH_s = pH_{actual,}$, water is balanced
pH_s $pH_{actual,}$LI = positive number, water is scale forming (nonaggressive)
$pH_s > pH_{actual,}$LI = negative number, water is not scale forming (aggressive)

It is important to remember that the LI value is not a quantitative measure of calcium carbonate saturation or corrosion. Because the protective scale formation is dependent on pH, bicarbonate ion, calcium carbonate, dissolved solids and temperature; each may affect the water’s corrosive tendencies independently. Soft, low-alkalinity waters with either low or excessively high pH are corrosive, even though this may not be predicted by the LI. This is because insufficient amounts of calcium carbonate and alkalinity are available to form a protective scale.

Table 16: Value of A for Water Temperature

Water Temperature (^{o}C)	A
0	2.60
4	2.50
8	2.40
12	2.30
16	2.20
20	2.10
25	2.00
30	1.90
40	1.70
50	1.55
60	1.40
70	1.25
80	1.15

Table 17: Value of B for Total Dissolved Solids

TDS (mg/l)	B
0	9.70
100	9.77
200	9.83
400	9.86
600	9.89
1000	9.90

Table 18: Value of C and D for Hardness and Alkalinity

Calcium Hardness or Total Alkalinity (mg/l as $CaCO_3$)	C1 or D2
10	1.00
20	1.30
30	1.48
40	1.60
50	1.70
60	1.78
70	1.84
80	1.90
100	2.00
200	2.30
300	2.48
400	2.60
500	2.70
600	2.78
700	2.84
800	2.90
900	2.95
1000	3.00

[1] Factor C is the logarithm (base 10) of the calcium hardness expressed in mg/L

[2] Factor D is the logarithm (base 10) of the total alkalinity expressed in mg/L

4.13.4 Removal of Lime Scale Deposit

Buildup of calcium in pipes and plumbing fittings can cause them to clog partially or completely, making them partially or totally dysfunctional. Some metals that pipes are made up of facilitate the buildup of calcium deposits. When such a case occurs, chunks of this calcium buildup can break off and clog the pipes, causing the pipes to become completely dysfunctional and even causing them to develop cracks and leak. If such deposits occur on your bathroom faucets or other fittings or tiles, they look ugly and dirty, no matter how many a times you clean it. If the water you are using has a high content of minerals in it, it will take as less as a month for the calcium deposits to build up in and around the pipes and plumbing fittings. Thus, the importance of cleaning the plumbing fittings and other pipes regularly cannot be over emphasized. There are various ways of removing lime scale deposited in the pipes and plumbing fixtures.

(a) Descaling Agents

Descaling agents are used to remove scale. A descaling agent or chemical descaler is a chemical substance used to remove lime scale from metal surfaces in contact with hot water, such as in boilers, water heaters, and kettles. Descaling agents are typically acidic compounds such as hydrochloric acid that react with the alkaline carbonate compounds present in the scale, producing carbon dioxide gas and a soluble salt. Strongly acidic descaling agents are often corrosive to the eyes and skin. Notable descaling agents include citric acid, formic acid, glycolic acid, hydrochloric acid, phosphoric acid, sulfuric acid and acetic acid.

(b) White Vinegar

Vinegar's acidic composition is a powerful tool in decomposing lime scale build-up. The easiest and cheapest method is to flush a concentrated solution of white vinegar through the inner surface of the pipes to cause the buildup to melt and fall off, which can be cleaned later with a brush that has a long handle.

(c) Water Softening

Water softening is the most common method of treating hard water. It works by a fairly simple chemical process – swapping the calcium (which forms lime scale) for sodium, (which is more likely to stay dissolved).

- As water enters the domestic system, it passes through an ion exchange column filled with thousands of tiny beads of resin.
- This resin has charged sodium attached to its surface and it swaps this for the more reactive calcium and magnesium as water flows over it.
- The resin can continue to do this indefinitely as long as it is washed through with salt water every so often to wash off the calcium and magnesium and replace the sodium. Most modern columns will automatically rinse themselves if regularly provided with salt.

(d) Water Conditioning

Water conditioning follows a far more hi-tech route. A water conditioner is attached to the pipes to expose the water to a low level magnetic or radio field. This field causes tiny impurities in the water to clump together. This will reduce lime scale because these clumps form a better surface for the dissolved minerals to stick to than the inner surface of the pipes, making them more likely to stay suspended in the water. If the water is left to stand, the minerals can form a kind of white sludge called 'soft scale', but this only accumulates in water tanks and is far easier to remove than a rock-hard layer of lime scale.

4.13.5 Prevention of Lime Scale Deposit

The removal of lime scale deposit is practiced once lime scale deposit has occurred. The removal methods of lime scale deposit are feasible only in household scale. In municipal or small town water supply systems, the lime scale deposit can occur at many places completely blocking the water flow. The public water supply system should try to achieve a system where lime scale deposit is prevented. This can only be possible by an appropriate design of water treatment system considering the various factors that leads to lime scale deposition. This is indeed a challenging task as it has not been yet practiced such treatment system in Nepal.

The water treatment plant consisting of following treatment processes is proposed for the prevention of the lime scale deposit in the pipelines.

(1) Aeration
(2) Lime soda softening
(3) Rapid mixing, flocculation and sedimentation
(4) Recarbonation
(5) Sedimentation
(6) Rapid sand filtration
(7) Disinfection

The treatment processes is schematically shown in Figure 45.

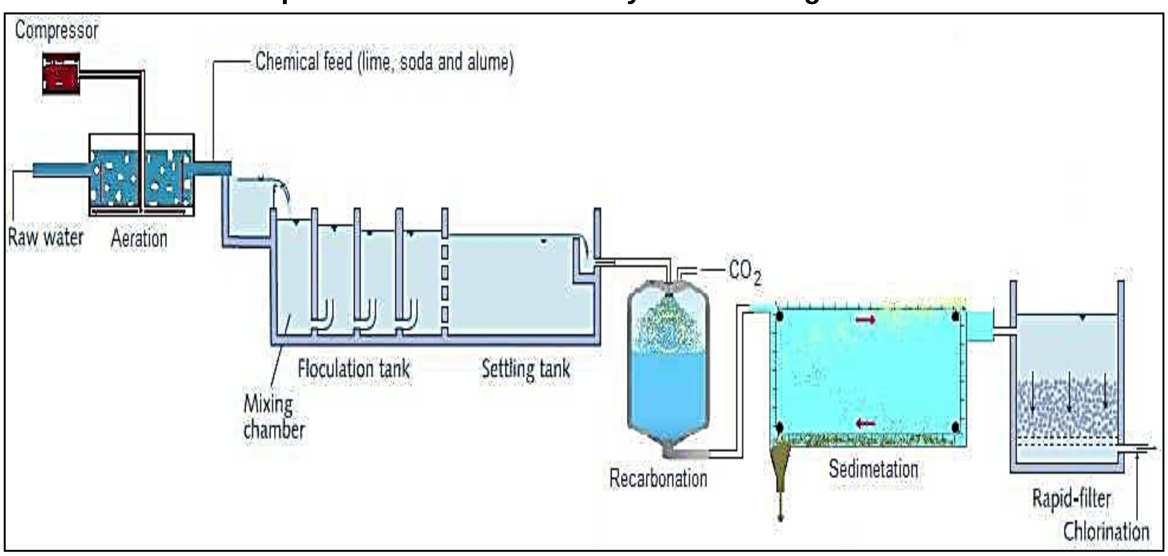


Figure 45: Treatment Process for Prevention of Lime Scale Deposit

The approach in the treatment should be to achieve the equilibrium condition of bicarbonate and carbonate with Langelier Index of nearly zero. The proposed treatment process should be applied only after verifying its suitable in pilot/laboratory scale testing.

4.14 Sludge Treatment and Disposal

Water treatment sludge is produced in the production of drinking water. The term “water treatment sludge” covers all wastes produced during treatment of water in a water processing plant. It is rather difficult to define the water treatment sludge in more detail as the water treatment sludge comprises both the sludge (this means, the real waste) and the wastewater. It is impossible to prevent the production of water treatment sludge. The composition and properties of the water treatment sludge depends typically on the quality of treated water as well as on types and doses of chemicals used during the water treatment. Depending on the quality of the treated water, the water treatment sludge contains suspensions of inorganic and organic substances. Typically hydrated alumina oxides and iron oxides are present depending on coagulants used for the treatment. Most of the water treatment sludge is made of alumina sludge.

Depending on the place of origin, the water treatment sludge can be divided as follows.

1. Rough trash caught in trash-racks. The character is really various. This sludge contains little water only. The quantity of this trash is very low.
2. Floc suspensions of iron and alumina oxides from thickening sections in settlement tanks and clarifiers. This sludge contains much water.
3. Filter washing sludge (the washing wastewater). This sludge is flocculent with small flocs. The settling velocity is low.
4. The sludge from removal of iron and manganese from ground water. Since lime is fed often and the sludge is easy to thicken thanks to high dehydrating of iron oxides, this sludge is easily processable. The quantity of this sludge is low and disposal is rather simple.
5. De-carbonization sludge. Sediments are more compact than flocculent sludge. The de-carbonization sludge is sometimes used as a fertilizer in the agriculture and does not represent a major issue.
6. Polymeric flocculants clarification sludge. This sludge contains typically suspended and colloidal particles contained in the treated water. Since fed quantities are rather low, the contents of the polymeric flocculants is low too.

Typical coagulants in the water treatment include aluminum salts and iron salts. The most popular is aluminum sulphate and aluminum sulphate modifications or combinations, such as chlorine aluminum sulphate. Ironic sulphate is the most frequently used coagulant from among iron salts. Therefore, the main component of the water treatment sludge is alumina and iron hydroxides produced in hydrolysis during the coagulation.

Solids generated in the water treatment operations consist of all suspended solids in the influent, plus all chemical agents added that produce precipitates and these solids settle in the sedimentation tanks. The sludge formed in the sedimentation tanks must be removed from time to time, treated and disposed of satisfactorily. The solids concentration in the sludge from sedimentation tanks will be about 0.5 to 2.0 percent.

Since the sludge collected from the sedimentation tanks contains about 98 to 99.5% water, it is necessary that the water content in sludge is reduced so that it can be transported to sludge disposal sites at a lower cost. The dewatering of the sludge reduces its volume drastically. The sludge treatment mainly refers to the dewatering of the sludge. There are three major treatment methods for water treatment plant sludge:

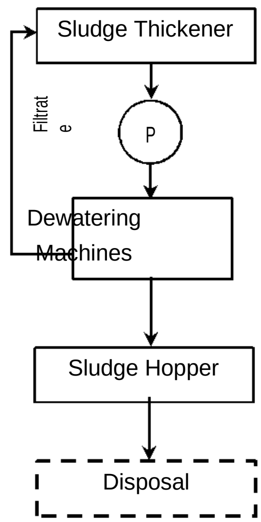
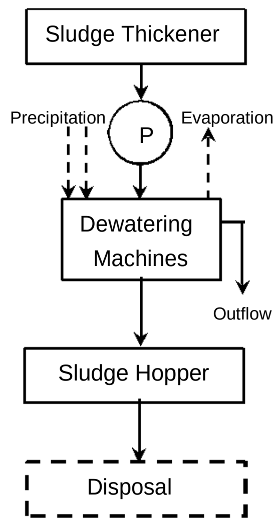
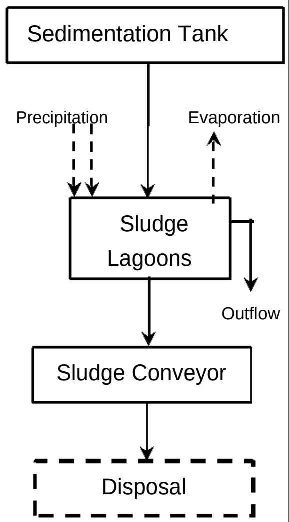
- lagoons
- thickeners + sludge drying beds
- thickeners + sludge dewatering machines

In general, sludge lagoons require a large land area, require less mechanization and provide easy operation. Sludge dewatering equipment requires less land area and provides good performance, but with greater operation skills required. Processes, principles and components are compared in Table 19.

(a) Lagoons

Lagoons are the simplest form of dewatering, requiring the least operator attendance. Dewatering is achieved by decanting clarified water to the level of the settled solids and by evaporation. Some drainage also occurs. With respect to lagoons, there is a practical difficulty in most of the places in Nepal, in that, for only eight months of the year evaporation exceeds the rainfall. Average figures of evaporation less rainfall indicate that from May to September, evaporation does not exceed rainfall, and in July, rainfall exceeds evaporation by over 400 mm

Table 19: Comparison of Sludge Dewatering Processes

Process / Item	Mechanical Dewatering (Filter Presses)	Solar Drying Beds	Sludge Lagoon
Process			
Principle	Mechanical equipment used for dewatering water treatment plant sludge includes filter presses, belt presses and vacuum filtration, and filter presses are the most popular. Filter Presses use a series of rectangular frames with a filter cloth spanning the edges of the frame. A hydraulic drive locks the plates together at the beginning of a cycle, and slurry is pumped into each chamber formed by the presses, and the pump maintains pressure in the chambers.	Paved shallow basins rely on evaporation to separate solids from water. Sludge storage facilities must be provided for periods when climatic conditions prevent effective dewatering. Air-drying processes are less complex, are easy to operate, and require less energy to operate than mechanical systems. They require a large land area, the operation depends on climatic conditions, and they are labor intensive.	Lagoon can be used for storage, thickening, dewatering, or drying. The lagoon process involves discharging residuals into a large hole in the ground, and the solid will be retained there for a long period of time. Solids eventually settle to the bottom, and liquid can be decanted from various points and levels in the lagoon. Evaporation may also be used in the separation process if the residuals are to be retained in the lagoon for a long period of time.
Components	- Sludge Thickeners - Filter Presses - Sludge Tank - Sludge Pumps - Air Compressors - Air Tank - Sludge Hoppers - Sludge Conveyors	- Sludge Thickeners - Sludge Pumps - Drying Beds - Sludge Conveyor	- Sludge Lagoons - Sludge Conveyor

De-Watered Sludge	55 to 65 % Uniformed thickness and moisture content	60 to 75 % Irregular shape and moisture content	60 to 75 % Irregular Shape and moisture content
Land Requirement	Small	Medium	Large
Construction Cost	Civil Works Large M/E Works Large	Civil Works Small M/E Works Small	Civil Works Medium M/E Works Small
Operation and Maintenance Cost	Manpower: Small Daily: Small Periodically: Small Electricity: Large Consumable: Large	Manpower: Medium Daily: None Periodically: Large Electricity: Small Consumable: Small	Manpower: Medium Daily: None Periodically: Large Electricity: Small Consumable: Small
O & M	Difficult	Easy	Easy

Land is at a premium at the site, and it is, therefore, desirable to reduce the area that would be required for any sludge drying system. The amount of land needed for sludge lagoons varies greatly depending on the presumed concentration of residue from the sedimentation tanks transferred to the lagoons. The use of a gravity-assisted mechanical thickener should be able to greatly increase the concentration of this sludge before sending it to the lagoons.

(b) Drying Beds

Drying beds could be sized for a drying period of about 4 months. Because the period available is only eight months there would again be difficulty in effectively drying the sludge. For drying beds to operate effectively, the dried sludge would need to be much drier than that from lagoons. It is possible that up to 60 percent solids could be achieved. Drying beds would have a higher construction cost than lagoons and would also require more attendance.

(c) Mechanical Dewatering

Thickening of sludge through purely mechanical methods includes the use of filter presses and centrifuges. For satisfactory operation, such systems require skilled operator attendance and complicated, and frequent, maintenance.

The quantity of the water treatment sludge is rather high. The water treatment sludge is placed mostly in landfills. In some countries, for instance in the Netherlands, about 25 per cent of the produced water treatment sludge is re-used. Efforts exist to recover alumina from the water treatment sludge during the water treatment or to use the alumina sludge for wastewater treatment or as a secondary raw material. The iron sludge can be also used for wastewater treatment. It is also possible to use the water treatment sludge in production of cements. Investigations are being carried out into the use of the water treatment sludge in agriculture and forestry.

It is still an issue to choose a disposal or liquidation method for the water treatment sludge that would be reasonable in terms of technology and economy. According to environment protection regulations it is required to minimize the quantity of wastes produced. If possible, the wastes should be re-used or processed as secondary raw materials as much as possible.

Unless the reuse of water treatment sludge is proven economically feasible, the disposal of sludge should be continued in landfill sites. The volume reduction of sludge is, however, highly recommended by reducing its water content so that the transportation and handling costs of its disposal are reduced.

5. GROUNDWATER ABSTRACTION AND DEVELOPMENT

Groundwater is the water occupying all the voids within a geological stratum. Groundwater is a part of the hydrological cycle and its recharge and discharge is a continuous phenomenon in the earth.

5.1 Groundwater Hydrology

Groundwater hydrology is the science of the occurrence, distribution and movement of water below the surface of the earth. Groundwater occurs in many types of geological formations known as aquifers.

5.2 Aquifers

Geological formation known as aquifers is the most important part in groundwater study. Formations that contain sufficient saturated permeable material to yield significant quantities of water to wells and springs are known as aquifers. Groundwater reservoir and water bearing formation are commonly used synonyms for aquifers.

5.3 Types of Aquifers

Unconfined, confined, leaky perched and idealized aquifers are the main types of aquifers. Soil particle size and their representative values of porosity should be considered while tapping the aquifers. Specific yield of aquifers depends on grain size, shape and distribution of pores, compaction of formation and time of drainage.

5.4 Geological Formation as Aquifers

Aquifers yield significant quantities of water from geological formation. Many types of formations serve as aquifers. The key requirement of aquifers is the ability to store water on the formation pores.

Alluvial Deposits are the main geological formation for groundwater development in Terai and Mid-Valleys.

Almost 90% of developed aquifers are in unconsolidated rocks. Site specific categorization of aquifers is very important. Depositional process, resulting strata, water courses, abandoned or buried valleys, plains and intermontane valleys of the area (region) should be clearly understood. Limestone, sandstone, volcanic rock, igneous and metamorphic rocks also serve as aquifers.

5.5 Groundwater Movement

Groundwater moment is governed by Darcy's Law which states that flow rate through porous media is proportional to the head loss and inversely proportional to the length of the flow path. Permeability and transmissivity of aquifers should be clearly understood which is controlled by aquifer thickness and inter-granular velocity. Groundwater reservoir(aquifer) is not a pond of water; it is water in the interstices of aquifer (formation).Water moves very slowly in the inter-granular path of the geological formation. Velocities values of 2m/year to 2 m/day are considered normal. Pumping test performed in the groundwater well provide information on groundwater movement and determines various hydrological parameters e.g. Q (discharge), SWL (Static water level), DWL (Dynamic water level), s (Drawdown), Specific yield, T (Transmissibility), K (Permeability), S (Storage coefficient), etc.

5.6 Well Design

Always be prepared for providing justification on your design. Groundwater aquifer is not like water pumping from a pond. While designing groundwater well one should not be over ambitious in the discharge rate. A well is a costly structure and the backbone of the project. Negligence in any process and work item may result project into failure. Selection of well construction material is very important. Analysis of water (corrosive, non-corrosive), well casing diameter, wall thickness and well depth need to be considered during the well design.

Designer should think for longer life time of project and should not compromise with inferior quality of materials. Mild steel slotted screen can provide maximum 10% open area, therefore shall be discouraged to use. Instead, SS304 stainless steel screens (20 to 25 % opening area) with continuous slot shall be encouraged to use in all the cases.

Sacrificial anodes shall be proposed in cases where dissimilar materials are used in the design. Aquifer thickness is very much important in well design, screening 100 percent of the aquifer is not recommended.

For well yield design optimum screen entrance velocity, clogging coefficient, screen diameter, length of the screen, percentage of open area should be considered. Screen should be placed so that the level of the draw down will be above the top of the screen as this permits contact with air and encourages corrosion. Screen opening should be 0.01 mm and more opening flared towards inside.

$$Q=\pi c v_s d_s L_s P$$

Where,

Q	=	discharge of the well, m^3/s
Vs	=	Optimum screen entrance velocity, m/s (generally 5 to 10 cm/s)
C	=	clogging coefficient (% of screen clogged
ds	=	Screen diameter
Ls	=	Length of screen, m
P	=	Percent of open area of screen (as per screen type)

5.7 Development Process

Least attention is given in the development process. Efforts should be made to open maximum pores of aquifers in the process of development. The process of development needs to be carried out serially.

A new well is developed to increase its specific capacity, prevent sanding, and obtain maximum economic well life. The removal of finer material from the formations surrounding the sell screen is quite necessary to obtain the full potential yields. The importance of well development cannot be underestimated and too often development is not carried out adequately in anew well.

Development procedures include inner and outer washing of tube well assembly by clean water and removal of drilling mud by diluting. The process is the first step of the well development which deposits finer material of formation inside the well assembly through screen sections.

These deposited materials are removed by bailer from well bottom. Bailing procedure agitates the formation and helps to rush in finer materials inside the well. The process is repeated until the whole deposited finer materials are not removed from well bottom.

After completion of bailing process the well is developed by suitable air compressor connecting to an air pipe into the well. Air creates a powerful surge within the well, first increasing and then decreasing the pressure as water is forced up the discharge pipe. This process loosens the fine material surrounding the screens. The loosened material brought into the well by continuous air injection creating an airlift pump.

This operation is continued and repeated until sand accretion becomes negligible. Air compressor development is required for prolonged period of time.

If the well yield is not satisfactory after above development process and there is still doubt about the mud cake formation on the wall of the drill hole the well needs development by chemical treatment.

Development method generally adding polyphosphates (sodium hexametaphosphate @ 15 to 25 Kg/m^3 of water in well for 24 hours) to water in the well will aid the development process. This compound act as deflocculants and dispersants of clay and other fine grained materials. The mud cake on the wall of a hole and the clay fractions in an aquifer are to be more readily removed by this development.

5.8 Pumping Test

Step drawdown test of the well should be carried out at least in 3 steps for the period of minimum 4 hours each step.

Specific yield has to be determined. Continuous pumping test and pumping equipment design should be based on the step drawdown tests. Continuous pumping test shall be performed for considerable length of time. All possible hydro-geological parameters shall be determined from the tests performed.

5.9 Drilling Methodology

Direct rotary, reverse rotary, cable tool (percussion) and pneumatic (air) methods of drilling are normally used for water well construction. Proper selection of drilling equipment and its suitability to the project scheme should be clearly defined. Drilling machine, equipment have their own importance and limitation to work in different terrains.

Direct rotary method of drilling is good for alluvial formation and suitable for greater depths and to penetrate grain size up to coarse gravel. This method is suitable in high pressure artesian areas. Reverse rotary method of drilling is suitable in soft areas, non-artesian areas and depth up to 150m.

Pneumatic drilling or DTH (Down to the Hole) is suitable in hard, compacted, consolidated rocks. Depth and diameter of drilling by pneumatic method is limited diameter up to 200 mm. The work progress is very fast but electrical logging and gravel packing work cannot be carried out.

DTH method of drilling do not need screen or slotted pipe rather the blind pipe is perforated with welding rod. The discharge from DTH is generally limited to 10 lit/sec. This method is not suitable in clay and sand area. The bottom of the well is open.

5.10 Cementing Wells

This consists of placing cement grout between the well casing and the hole from the surface to the level when the casing.

5.11 Guide Pipes

Drilling in the areas where the top 5 to 10 m soil is loose. It is recommended to install 22” diameter mild steel guide pipe of length of 6 to 12 m. This prevents the well from collapsing during drilling. This can be removed after completion of drilling and cement grout is filled up to this depth.

6. ELECTRO-MECHANICAL EQUIPMENT

Pumps for deep tube-wells are designed mainly with the following basic objectives:

- Safe working conditions for operation and maintenance personnel;
- Easy accessibility and operation;
- Long term reliability;
- Minimum capital and operating and maintenance cost;
- Contamination free water;
- Unobtrusive location;
- Energy efficiency.

All electromechanical components shall be designed for the design lives of 15–30 years. For easy maintenance, equipment should be of standard types and interchangeable with or preferably identical to each other, if two or more units exist.

Though there were trends in the past to keep a standby tube-well, only a standby submersible pump with necessary accessories shall be advised. Submersible motor pump installed in deep tube-well lifts water either directly to overhead service reservoir or to a water treatment plant from where water is finally lifted to the service reservoir either with submersible or above ground centrifugal pump. Appropriate shelter or Pump house (room) is necessary for centrifugal pumps. However, pump house is not essential for submersible pumps when all weather proof (outdoor type) motor control panel is used.

Electric power necessary for prime mover of the pump is obtained from public authority through 3 Phase 11 KV transmission lines, which is converted into TPN (four pole device with 4th pole as neutral) 400V and 220V with a distribution transformer. Provision of a suitable capacity standby diesel power generator shall be made in each pumping station. Capacity of the distribution transformer and diesel power generator should be sufficient for pumping equipment, treatment plant, compound and house lighting and other general uses. Electricity is distributed through a power distribution box consisting of individual circuit breaker for each unit.

When two or more pumping units exist, individual suction and delivery pipelines with necessary accessories should be provided to each unit. However a common delivery line with combined capacity may be used to transport water to final destination.

The Design Work of Electromechanical Part of a Drinking Water Pumping Station is to

Determine:

- Volume of water collection tank in pumping stations
- Type and diameter of water conveying pipeline. Maximum pressure and thrust load in pipeline. Size and shape of thrust block wherever is necessary.
- Type, number and specification of pump set and accessories.
- Type of electric power supply and electrical control devices
- Type of installation of electro-mechanical equipment, and
- Cost of operation and maintenance

6.1 Types of Pumping Stations

On the Basis of Types of Water Source Drinking Water Pumping Stations may be classified into Three Categories:

1. Surface Water Pumping Station
2. Groundwater Pumping Station
3. Subsurface Water Pumping Station

6.1.1 Surface Water Pumping Station

A surface water pumping station consists of a water collection tank receiving water from a surface source like, rivers, springs, streams, lakes etc. Pumping equipment with necessary accessories is installed in a pump house and water is pumped through a transmission main pipeline to a service reservoir or a treatment plant. Usually, horizontal centrifugal pumps are used for surface water pumping.

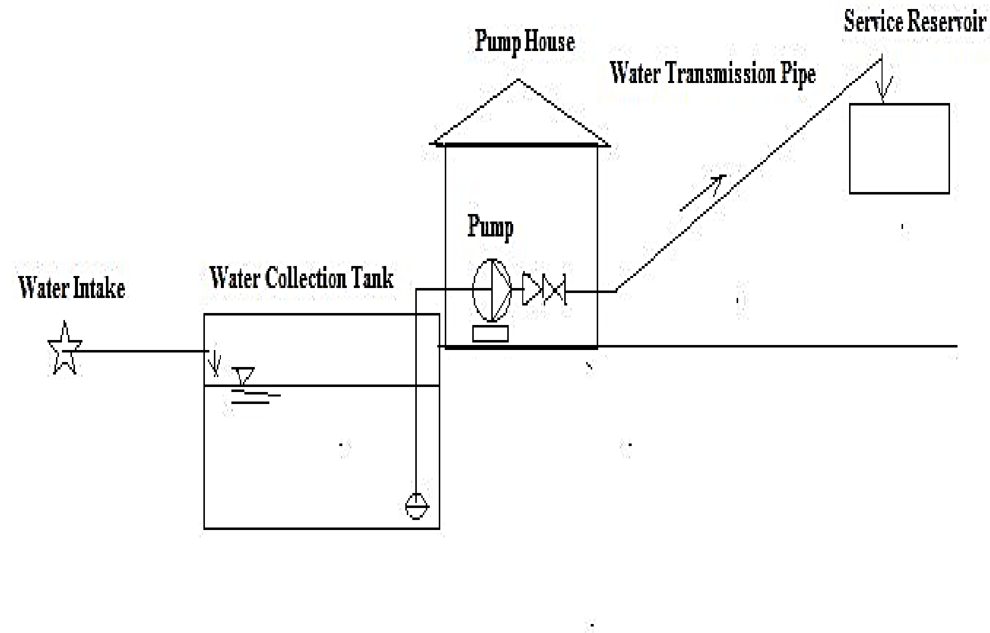


Figure 46: Surface Water Pumping Station

6.1.1.1 Horizontal Centrifugal Pump

Where space is not limitation and if the water level is within the permissible suction level of the pump, possibility of installing horizontal type of pump could be explored. A centrifugal pump-set consists of a centrifugal pump coupled with a suitable prime mover (electric motor) on a common base plate. Unlike a deep well submersible pump- set it needs a shelter/ pump house

Very often the capacity of pump does not comply with the required discharge. This means that the pump will have to be stopped occasionally and restart later. The number of starts must be limited for two reasons:

- The relatively high start-up power is required.
- The overheating of motors must be prevented.

For this reasons the number of starts per hour must be limited to few times.

The sump capacity may be calculated with the formula:

$$V = \frac{3600(Q_p\,Q - Q^2)}{s.Q_p}$$

In which

V = the sump volume (or reservoir volume) between switch-on and switch-off levels (in m^3);

s = the no of starts per hour;

Qp = pumping rate (in m^3/s);

Q = water inflow rate (in m^3/s);

The required volume is a minimum if the inflow equals half the pumping rate, in which case V = 900. Qp / s.

$$\boldsymbol{V = \frac{900.Q_p}{s}}$$

6.1.1.2 Pump House Design and Construction

Some important notes:

- Pump house should not only be accessible during the construction phase, but also during the execution of operation and maintenance. Layout should be so that no staff has to walk close to the high tension portion of the electric line.
- The pump house should be constructed in such a height that the mechanical and electrical equipment must be free from flooding.
- Special measures will be required and structural stability will have to be assured for pump
- house constructed near to a stream or a slope.

  Sufficient space for mechanical and electrical equipment in the pump room Pump room should have space for the pump operator to watch the equipment during operation and working space during maintenance. The flooring of the pump house should be strong enough and should not be damaged during repair of the machine. There should be sufficient space to move between them during maintenance purpose, but no unnecessary empty place. All space should be well lighted. The door of the pump room should be large enough and should open outwards to allow passage of all parts of the installation as well as to use it as an emergency exist. Drainage opening must be provided in the pump room.

In a pump house at least one set of pumping equipment should be installed as stand-by unit. The floor area of the pump room depends on the number and size of the pumping equipment. Hence, design of the pump house should be done after deciding the number of the pumping units to be installed and knowing the size of the equipment.

An installation that requires the total capacity Q can be equipped with two pumps, each with capacity 1/2Q, or three pumps, each with capacity 1/3Q. If one pump can no longer operate, the installation can still work at 50% or 66% of the total capacity, and the pumping station will not be completely idle. In order to maintain full capacity when one pump is out of order, two pumps, each with capacity Q, three pumps each with capacity 1/2Q, or four pumps, each with capacity 1/3Q must be installed.

Space for the installation of additional pump set in next phase (when it is necessary to increase capacity of pumping) should be provided.

- Construction of a pumping station may have effect on environment. Hence, following should be considered:

  (a) Existence of schools, hospitals and residential complexes
  (b) Noise, vibration, pollution and similar regulations
  (c) Effect on natural environment, scenery, animals and plants
  (d) Recreation areas, parks etc.

When it is required to lift water to very high head it becomes necessary to construct multi stage pumping stations. Division of heads between the pumping stations should be done in such a way that the equipment and their accessories and pipe materials working in pressure should be easily available in the nearby markets.

6.1.1.3 Size and Shape of Pump Room

Size and shape of pump room depends on:

(a) Method of pump installation and,
(b) Layout of pumps

6.1.1.4 Method of Pump Installation

The method of installing the pump is generally determined in accordance with the site situations, or the terrain and the suction water level. There are two typical systems: minus back and plus back. Under the minus back, pumps are installed above the suction water level and under plus back, pumps are installed below the suction water level. The plus back system is more advantageous. Hence, plus back system is to be selected, wherever possible.

Positive Suction Head Negative Suction Head

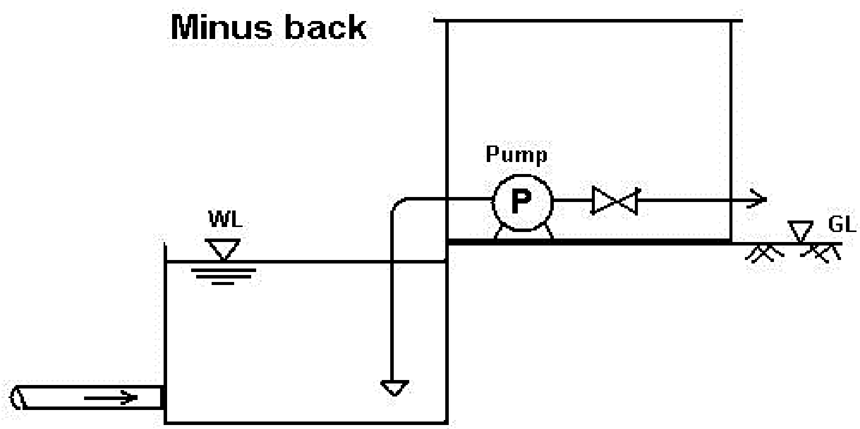


Figure 47: Surface Water Pumping Station

On deciding the suction head of a pump and suction pipe diameter the phenomenon of **cavitation** should be counted for which available **NPSH**, the net positive suction head is determined by using relation:

$$\mathbf{h_{sv} = Pa - Pv + Hs - HLs - 1.5}$$

Where,

hsv : Available NPSH
Pa : Atmospheric pressure, m
Hs : actual suction head, m
HLs : loss head of suction pipe, m
Pv : saturated vapor pressure, and
1.5 . : is an allowance

The NPSH available at the site of pump installation (site specific) should always be greater or at least equal to the NPSH required (the characteristic of the pump specified by the manufacturer). Available NPSH is the sum of Barometric pressure and the static head on the pump inlet, less the losses in the pipe and fittings and the vapour pressure of the water.

Table: Barometric Pressure

Altitude, m	KPa	m of water
0	101	10.33
305	98	10.00
610	94	9.60
1220	88	8.90
1830	81	8.29

Table: Vapour Pressure of water at different temperature

Temperature $^{0\,C}$	KPa	m of water
21.1	2.5	0.25
26.7	3.5	0.35
32.2	4.81	0.49

Example:

Assume that a water pumping station at 500 m elevation uses pumps which require 30 KPa (3 m) positive suction pressure (NPSH) when delivering water at 30^{0}c. What is the allowable suction lift of these pumps if entrance and friction losses are 15 KPa (1.5m) ?

Solution:

Barometric pressure at 500 m from mean sea level = 95.4 KPa. Reducing this to account for possible low pressure circumstances,

Pa = 95.4 – 3.5 = 91.9 KPa

The vapour pressure of water at 30^{0}c (Pv) = 4.3 KPa

Now, NPSH avail (hsv) = 91.9 – 15 -4.3 –Ps

NPSH reqd = 30 = 72.6 – Ps

Ps = 42.6 KPa (4.35 m)

Hence, the available lift is 4.35 m. If the circumstances required a greater lift, either another pump would be selected, the entrance condition would be modified to reduce losses, or the pump would be moved to reduce the lift to that of which the pump was capable. As the available NPSH decreases the capacity of the pump decreases.

Cavitation: If the NPSH drops below that required by the pump design, the pressure within the eye of the impeller may be reduced to the vapor pressure of the water. If this occurs the water will vaporize and a mixture of vapor and water will enter the pump. In the extreme, the pump may lose its prime, A more common occurrence is continued flow at reduced capacity with bubbles forming in the impeller eye and being passed out along the impeller. As the bubble traverse the impeller they pass from a region of low pressure to one of high pressure and eventually collapse permitting high pressure water to strike the impeller. This collapse frequently causes pitting of the impeller surface and is always accompanied by a ratting or pinging noise. Cavitation can be avoided by selecting pumps which operate at heads and capacities close to those corresponding to maximum efficiency, by following manufactures' recommendations with regard to maximum suction lift and minimum discharge pressure and by not altering pump speed from that of the original design.

6.1.1.5 Layout of Pumps

In general two pumps of same capacity are installed in a pump station of drinking water supply project. Possible layout styles are as follows:

(1) Straight line arrangement
(2) Opposite arrangement
(3) Parallel arrangement

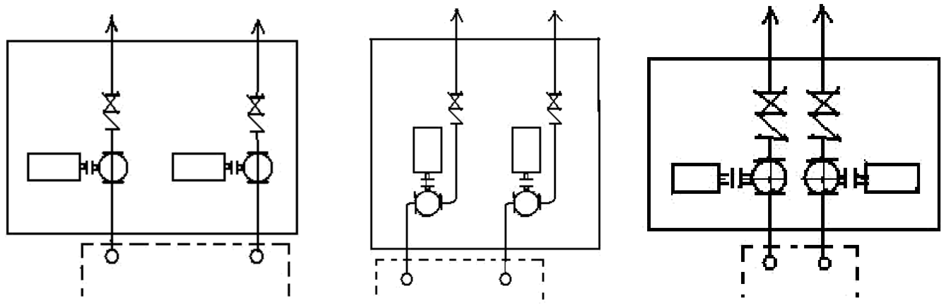


Figure 48: Layout of Pumps

Because of most simplicity and possible of smaller suction sump the opposite arrangement is chosen wherever is possible.

The minimum space between two adjoining pumps or motors should be 0.6 m for small and medium units and 1 m for large units.

A clear space of not less than 915 mm in width shall be provided in front of the switch board. In case of large panels, the recommendations of the manufacturer should be followed.

6.1.1.6 Pump set

Centrifugal pump set in general shall conform to ISO 9908-1993 or IS: 1520-1972. The pump may be volute or turbine type, single/multistage, directly coupled with its prime mover on a common base plate.

6.1.1.7 Pump Foundation

(a) Pumps are mounted on concrete foundations, which when located on soil should be carefully designed so that the total load (weight) of the pump set and foundation itself does not exceed the allowable bearing capacity of the soil.
(b) The weight of the foundation should be 3 to 5 times of the total weight of the pump set (in case of motor-driven pump).
(c) Directly coupled pump and motor should have common base plate and common foundation block, so that a slight sinking of the ground below pump or motor may not cause an error in level.

(d) When the foundation of the pump should be constructed on the upper floor of a building or any floor not supported by the ground, longitudinal center line of the foundation should coincide with the center line of the floor beam. If this is not possible, then each end of the foundation should be supported by existing beam even it may be long.

(e) Concrete mixing ratio of cement, sand and gravel should be made either with weight or volume, but weight method is considered more accurate in ratio 1:2:4.

6.1.1.8 Suction Piping Layout

(a) Each pump shall have independent suction.

(b) Suction piping should be as short and straight as possible

(c) Any bends or elbows should be of long radius.

(d) Where suction lift is encountered, no point on the suction pipe should be higher than the highest point on the suction part of the pump.

(e) When a reducer is used, it should be of eccentric type.

(f) When working on suction lift, the taper side on the reducer should be below the center line of the pump

(g) When suction lift is encountered, a foot valve is provided to facilitate priming. Foot valves are normally available with strainers. The net open area of the openings of the strainer should be minimum equal to three times the area of the suction pipe.

(h) When there is positive suction head, a sluice or a butterfly valve should be provided on the pump suction, for isolation. The sluice valves should be installed with their axes horizontal to avoid formation of air pockets in the dome of the sluice valve.

(i) When suction lift is encountered, a foot valve is provided to facilitate priming. Foot valves are normally available with strainers. The net open area of the openings of the strainer should be minimum equal to three times the area of the suction pipe. When there is positive suction head, a sluice or a butterfly valve should be provided on the pump suction, for isolation. The sluice valves should be installed with their axes horizontal to avoid formation of air pockets in the dome of the sluice valve.

6.1.1.9 Discharge Piping

(a) Discharge piping connecting to a common manifold or header shall be connected by a radial Tee or by 30° or 45° bend.

(b) Delivery line of each pump shall be connected to the main header.

(c) Each header shall be provided with check (N-R) valve, discharge valve, pressure gauge etc.

(d) The pipe network and valves within the pump house shall be adequately supported so as to avoid undue stress on the pumps.

(e) Flow velocity in discharge pipe shall be 0.5 – 1.5 m /s .When there is a possibility of increasing the pumping capacity, the lower velocity should be selected.

(f) Selection of material of construction of pipe shall be selected to withstand maximum pressure in the pipeline.

Maximum pressure in a pipeline occurs when happens water hammering during sudden stoppage or start of pump due to interruption electric supply which in many cases cannot be avoided. In such condition danger of pipe collapsing with low pressure or bursting with high pressure always exists. Working pressure of the pipe used shall be either more than the maximum pressure in the pipeline or there must be provision of preventing water hammering.

Use of heavy flywheel in case of centrifugal pump and pressure (surge) vessel in case of centrifugal as well as in case of submersible pump are in practice.

6.1.1.10 Auxiliary Piping

In case of minus back system permanent piping for priming with a suitable **priming** tank shall be provided. If there is a possibility of flooding the pump room dewatering pump shall be provided.

6.1.1.11 Spare Parts

Provision of spare parts required at least for two years of normal operation (as recommended by the manufacturer) should be made.

6.1.1.12 Tools and Equipment

Tools and equipment required for the easy operation and maintenance. In case of deep tube-well pumps lifting equipment like, chain pulley block set including at least 2 sets of riser pipe clamp is essential

6.1.1.13 Drawings and Operation / Maintenance Manual

Drawings indicating positions of well screen and pump setting depth, operation and maintenance manuals of all equipment present in the pumping station should be provided.

6.1.1.14 Operation Record and Maintenance History

Regular operation record and maintenance history should be kept in the pumping station. For this appropriate forms should be designed and operators should be properly instructed to fill them.

6.1.2 Submersible Pump for Surface Water Pumping

For surface water pumping when the pump station is located at an above ground storage facility, generally centrifugal horizontal pump is preferred. However, because of following reasons submersible motor pumps may also be selected for the proposed project:

1. Submersible motor pump requires less space. It needs no pump house with big space, small size room is sufficient for electric panels.

2. As the pumping stations are located in forest, needs protection against unauthorized human and animal activities which is possible when pump sets are installed in closed chambers which is possible for submersible motor pump.
3. Submersible motor pumps has noise free operation hence, are friendly to environment.
4. Operation of submersible motor pump is easy. Presence of pump operator during the total operation hour is not necessary.
5. As the submersible motor pump does not require periodic maintenance and it is easy to operate in comparison to the centrifugal pump is suitable for the local geographical situation.
6. Submersible pump set is easily available in local markets and are less expensive. It is easy to transport. Its installation work is not complicated and may be performed by a trained operator whenever becomes necessary.
7. Submersible motor pump sets are more efficient than conventional centrifugal pumps, consume less energy and hence, operate in low cost.

Depending on motor type, the pump can be installed either vertically or horizontally in a water tank. However, because the water tank is to be emptied and maintenance personal has to go inside the water tank contaminating it. Hence, installation of submersible pump set in a ground tank is not recommended. Without disturbing ground tank, submersible motor pump can be installed underground in a mild steel housing pipe. A water level indicator with reading gauge to be fitted in the tank and there should be a provision of chain pully stand centered at well head is to be installed.

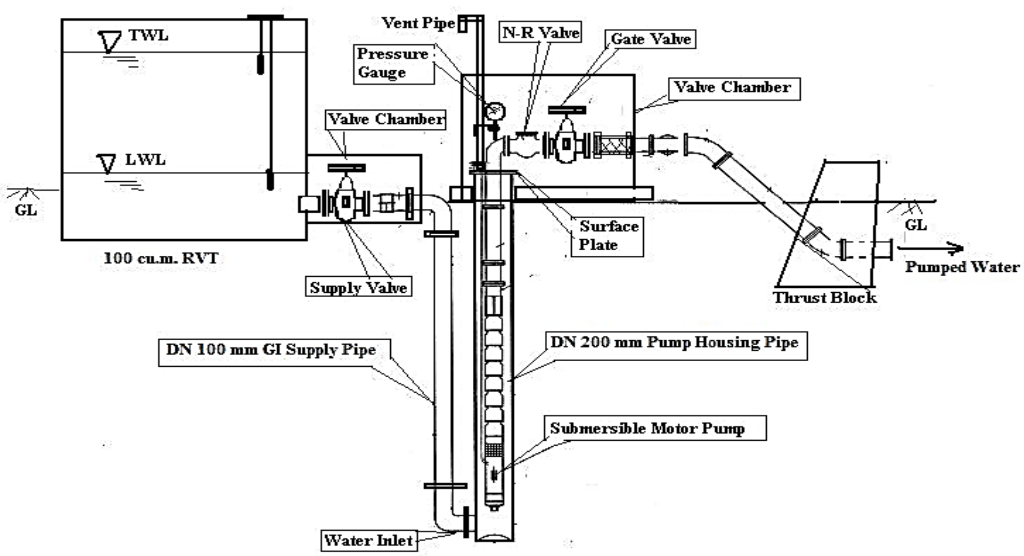


Figure 49: Submersible Pump Installation for Surface Water Pumping (Option 1)

This should be considered that the entry of water should be from the lowest portion of the pump housing pipe and size of the housing pipe should be so selected that the flow rate in it during pumping of water should not be less than 1m/s - 1.5 m/s. These provisions are made for the cooling of motor.

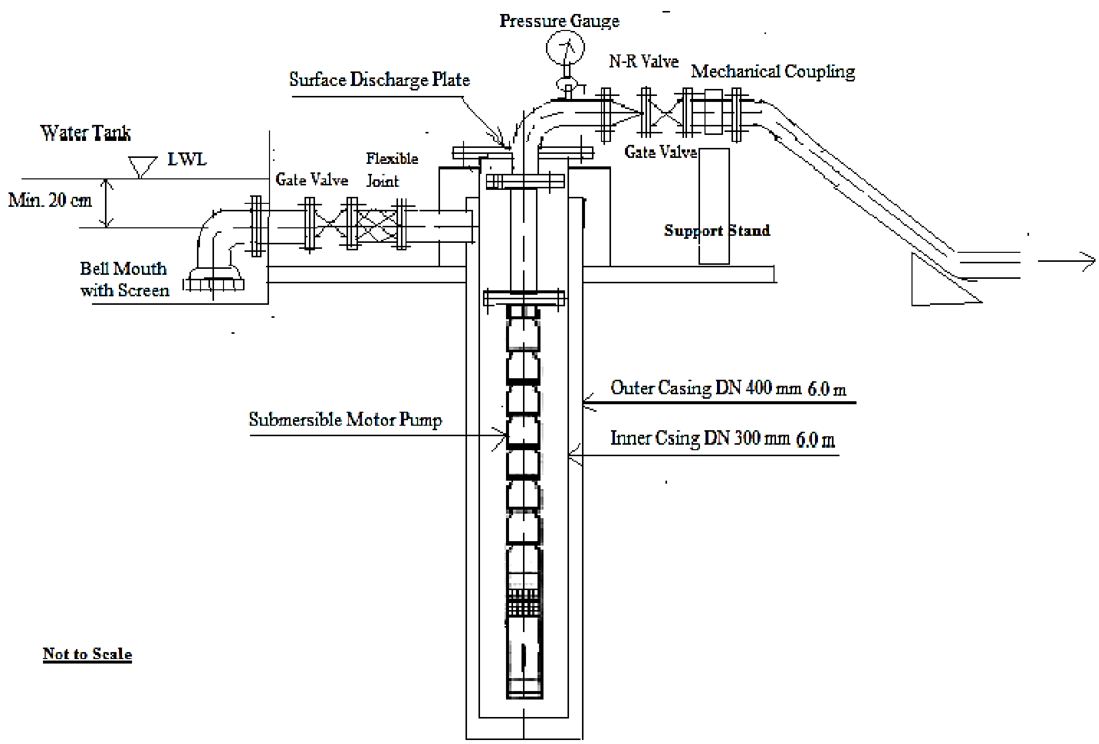


Figure 50: Submersible Pump Installation for Surface Water Pumping (Option 2)

Submersible motor pump is installed in a mild steel housing pipe of appropriate diameter. The pump housing pipe itself will be fixed underground close to the water collection tank of the station. Pump shall be operated semi automatically. Provisions for the protection of pump motor against over loading, dry running and single phasing is necessary.

6.1.2.1 Number of Pump Sets

One pump for normal use and one for standby with full capacity pumping rate may be installed in each station , however , it is suggested to install three pump sets each with ½ capacity pumping rate normally , operating two pumps in parallel and one remaining as standby because of following reasons:

a) It reduces initial starting current for motor which reduces overloading of transformer in each start.

b) In normal operation it reduces effect of water hammering in pumping main.

c) It requires less diameter accessories, reduces cost for replacement.

d) When one pump set is out of order there will be no reduction of pumping capacity and even if two pumps are out of order then also 50% production capacity is remaining and there will be no interruption of water supply. It gives more time for repair

6.1.3 Groundwater Pumping Station

A groundwater pumping station is consist of:

a) Deep tube-well;
b) Pump house (room) with lifting arrangements; Submersible motor pump and accessories and delivery pipeline.
c) Transformer station including high voltage switch gear;
d) Low voltage switches gear, switch board and control board
e) Standby diesel power generator and generator room

6.1.3.1 Deep Tube-Well

Designer of pumping station must collect detailed drawing of the deep tube-well showing total depth, size and depth of casing and housing pipe and length and position(s) of screen together with the following additional information:

- Static Water Level (SWL) below ground level (m, BGL);
- Capacity (safe yield) liters per minute or liters per second;
- Dynamic (Pumping) water level at rated capacity (m, BGL);
- Chemical analysis report of water, especially sand content (ppm), which shall not exceed 50 ppm for a normal pump.

6.1.3.2 Pump House

- Pump house should have sufficient height (min. 3 m) and space (min. 3m x 4m) to accommodate complete set of electromechanical equipment.
- The elevation at the top of the well casing should be above the existing ground surface, the normal flood level of any adjacent water body, and at least 0.15 m above the finished floor level of the pump house.
- A pump pedestal, raised at least 0.15 m above the finished floor elevation, should be provided to support the full weight of the pump.
- The weight of the pump and its discharge assembly should not be borne by the pump base and reinforced concrete floor slab.
- A hatch with minimum dimensions of 800 mm x 900 size should be provided in pump house roof directly over the well. The hatch may, if desired be a removal type skylight unit. To allow for pump removal, the well should be positioned 600 – 1200 mm from the outside wall, and be adjacent to an access road designed for heavy vehicle. Double entrance doors should be provided, which should open outwards and sized such that they are wider than the largest piece of equipment in the pump house.

- An I- beam or a sufficient diameter MS/GI pipe beam supported on two RCC pillars at least 1 m higher than the roof slab should be provided directly over the well which should be capable to bear at least 3 times the weight of complete pump unit and water in the column (riser) pipe. A chain pulley block should be hanged on the beam.

6.1.3.3 Operating Parameters of Submersible Pump

Submersible pump is used to lift water from a tube-well to a higher ground with the help of an electric motor. Pumping head, discharge and power are the operating parameters of a submersible pump.

Pumping Head

While pumping, a pump must overcome the total pressure head i. e. the summation of static, friction and velocity heads. Pressure is normally expressed as meter of water (1 m head is equivalent to a pressure of 9.81 kN/m^2)

Static head: When a submersible pump lifts water from a tube-well, the static head is measured vertically from the dynamic water level (static water level + drawdown depth) to the outlet of delivery pipe (expressed as hsd).

Friction head: The head required to overcome friction in the system (pipes and fittings) is the friction heads loss. This can be divided into friction losses on the suction side of the pump (expressed as hfs) and on the delivery side (expressed as hfd).

Velocity head: A certain amount of pressure head is needed to accelerate the water from zeroto its flow velocity, (hv = v2/2g). Being very less in most practical cases, this is neglected.

Residual head: Head remaining at the outlet of the delivery pipe 3m – 5m (expressed as hr)
Hence, total pumping head, H = hsd + hfd + hr

Capacity (Discharge / Pumping Rate)

Flow rate of pumped water against a certain pressure or “head” at a given speed is commonly expressed as m3/h, l/m or l/s.

To avoid sand/ silt pumping, capacity (pumping rate) of submersible pump should be less than the safe yield of the tube-well and entrance velocity of water through the tube-well screen should be less than 0.03 m/s.

Power

The power output of a pump (Pw, measured in kilo watts), sometimes called the water power, is given by:

$$Pw = rQH$$

Where,

Pw Pump Power in KW ‘ r unit weight of water in KN/m^3 (10 KN/m^3)

Q flow rate of water, m^3/s

H Total dynamic head including suction head, delivery head, friction head and velocity head, m is the operating pressure head against which the pump must discharge.

The required input power to the pump, Pp is given by

$$Pp = Pw / л$$

Where,

Pp Power input to the pump, KW

л Over all efficiency of pump and motor (Normally taken as 0.6)

Example:

Determine the water power, pump power and motor load for a pump system designed to deliver 30 l/s against a total system head of 50 meter. Assume both pump and motors are 80 percent efficient.

Pw = r Q H

= 10 KN/m^3 *($30*10^{-3}$)* 50 m

= 15 KW

Pp = 15/0.8 = 18.75 KW

Pm = 18.75/0.8 = 23.43 KW

In this case Efficiency л = Ep * Em = 0.8*0.8 = 0.64 say 0.6

Pm = r*Q8H/ л

= $10*30*10^{-3}*50/0.6$

= 25 KW

= 25/0.748 = 33 HP

Where η is the overall efficiency with which power from the prime mover is converted into water power. This will depend on power transmission efficiencies and pump efficiencies. Transmission efficiencies may be 90 - 95 percent for close coupled electric motors. The pump efficiency changes by the type of pump, capacity, head, speed and various other conditions and it is difficult to define in a general manner or to calculate by a simple equation. As an approximate criterion, above figure shows the approximate efficiencies obtained based on JIS and other literatures and also on the documents of actual results. To calculate roughly the power of a pump, this figure may be used.

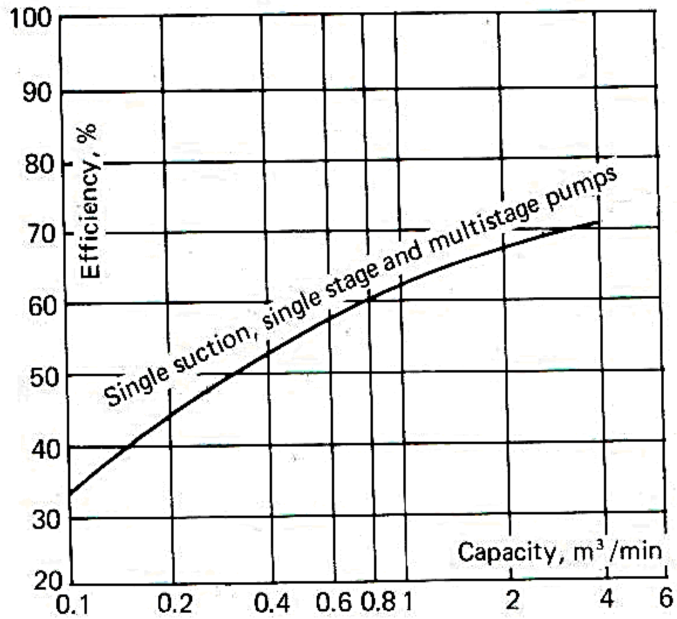


Figure 51: Efficiency of Pump Operating Point of a Pump in the System

When the static head is known and the pressure losses of the system at a given discharge is calculated, the pipeline or system characteristic or external head of the system can be plotted in a graph showing the relationship between the external head and discharge (Qs - Hs curve). This is done by plotting the pressure losses, which increase with the square of the discharge above the static head. If a similar Qp – H p curve of a pump is drawn in the same scale (generally obtained from the manufacturer), the pump will automatically produce a discharge that will remain constant at the point given by the intersection of external head and pump characteristic. This point of intersection is the operating point. Preferably a pump has to be chosen where the design point (max. efficiency) coincides with the operating point.

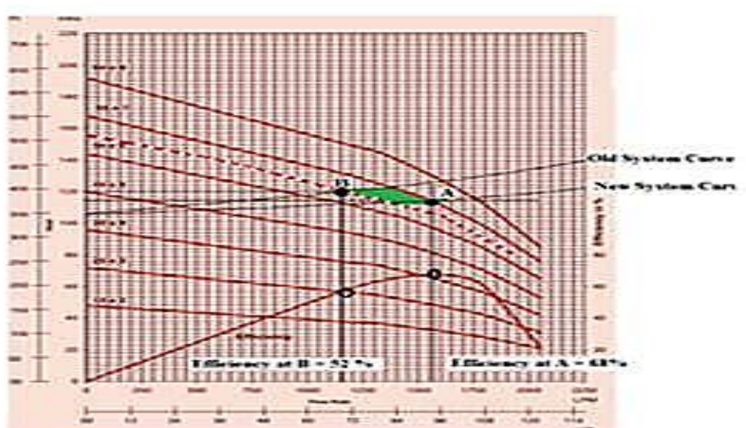


Figure 52: Efficiency of Pump

Special requirements of pump design, construction and performance of the pump specific to that individual site need to be taken care of pump selection for which it needs to be understood:

- Whether the requirement is in respect of constant capacity / flow rate at fixed head. In such a case, the pump is supposed to work almost on the stipulated duty point on head capacity curve and hence high value of efficiency at that point would be appreciated.
- Variation in capacity or flow rate with head / pressure remaining nearly constant: this is encountered in actual practice e.g., when a pump has to discharge into water supply mains of a colony/plant, under a nearly constant pressure. For such operation, flathead capacity curve would be desirable.
- Variable head with slight capacity variations: this is common situation, involving a variation in static head due to variation in water levels in different seasons, which should not be followed by considerable capacity variation. For such an operation, steep - head capacity curve would be desirable.
- The pump may also be required to be operated under severe adverse conditions of voltage and frequency. If so, a motor should be appropriately selected.

6.1.3.4 Parts of Submersible Pump

The pump in general shall be tolerance to ISO 9906 Class 2 (Indian shall confirm to IS: 8034 – 2002 or equivalent.

- Pump bowl: Material - Cast iron. The bowl unit shall be capable of withstanding a hydraulic pressure of equal to twice the pressure at the rated capacity or, 1.5 times the shot off head, whichever is greater.
- Impeller: Material – bronze/CrNi Steel/Noryl (PPO). Radial or mixed flow type
- Shaft: Material - stainless steel
  - Submersible Motor shall be confirmed to IS:9283 - 1995 (R2002) or equivalent

Direct On-line (up to 7.5 HP), Star-Delta (7.5 to 50 HP) and autotransformer/soft starter or variable frequency drive (VFD) started (above 50 HP), squirrel –cage induction type suitable for operation on 400± 10% V, 3 Phase A.C. 50 HZ

- The motor should have at least 10% margin at duty point as well as should not get overloaded in the entire range of operation
- The thrust bearing shall be of adequate size to withstand the weight of all rotating parts as well as the imposed hydraulic thrust. It shall have sufficient capacity to permit the pump to operate for short periods with discharge valve

Note: When sand / silt content in water is more than 50 ppm (up to 150 ppm) pump-set made of special materials like stainless steel is to be used.

6.1.3.5 Accessories and fittings

Following accessories and fittings are associated with submersible pump set:

a) Riser/ Drop (column) pipe
b) Surface discharge plate
c) Pressure gauge
d) Check (non-return) valve
e) Delivery (gate) valve
f) Submersible cable
g) Motor control panel

The accessories and fittings associated with submersible pump set are shown in Figure 47.

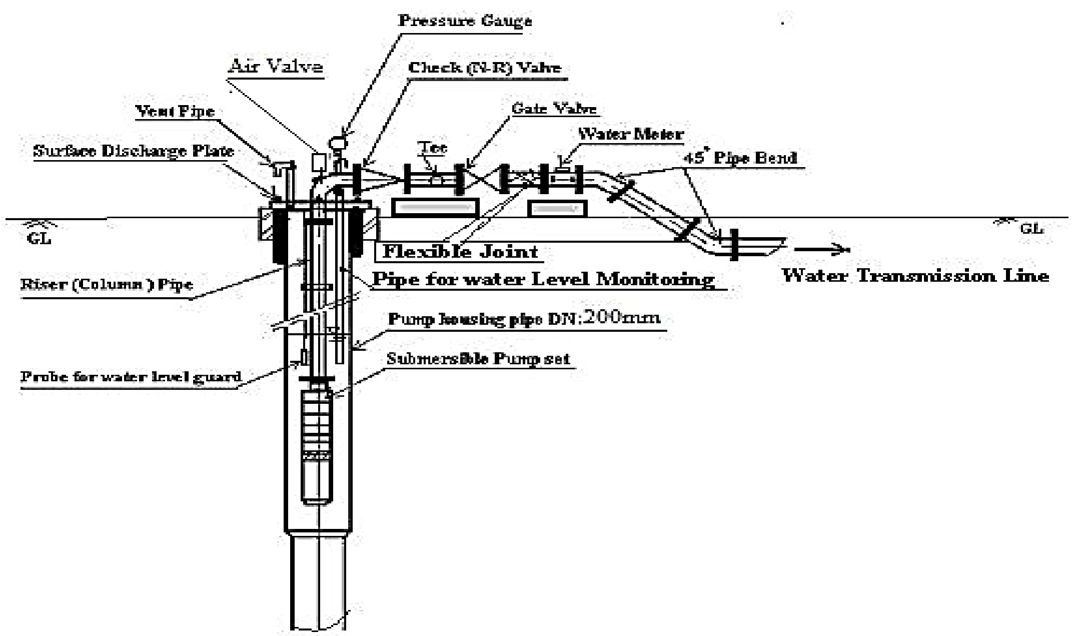


Figure 53: Submersible Pump Set – Accessories and Fittings

a) **<u>Riser / Drop (Column) Pipe</u>**

It is made of heavy class Galvanized steel. It may be ERW or Seamless. UPVC or any other plastic pipe is not recommended.

- Each riser pipe shall be 3 m in length except the top and lowest pipe which shall be 1 m long. The pipes shall be flanged ended with welded flange in each end. The Flange should be according to IS: 6392 -1971 for 1.6 N/mm2.
- Each flange should have two cuts for the cable entry with sufficient depth. The maximum diameter of the flange should be at least 20 mm less than the inner diameter of the well.

As the submersible pump set is lifted out of the well and reinstalled several times the nut, bolt used should be made of stainless steel.

When diameter of water well does not allow flange joint special tapered threaded galvanized iron riser pipe with seamless coupling may be used.

Generally suitable column pipes both end flanged pipes for different well diameters are illustrated below:

Diameter of Housing	Diameter of column pipes
300 mm (12”)	150 mm (6”)
	125 mm (5”)
250 mm (10”)	100 mm (4”) or lower
200 mm (8”)	80 mm (3”) or lower
150 mm (6”)	50 mm (2”) or lower

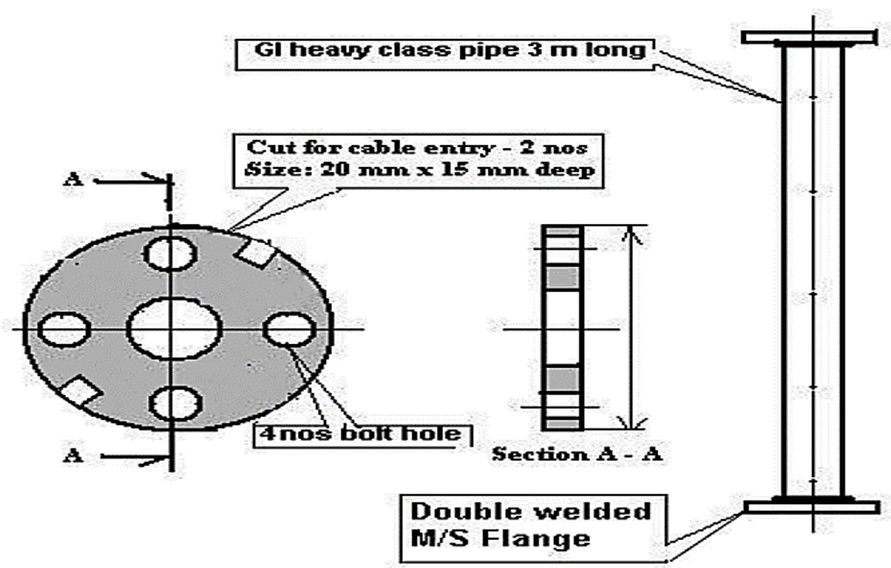


Figure 54: Riser Pipe

b) **Surface Discharge Plate**

Surface Discharge Plate also acts as cover to the tube well. It is made of MS blind flange corresponding to IS: 6392-1971 having sufficient strength to support entire weight of the pump set. The surface plate consists of a pipe bend of size equal to column pipe size. The discharge bend has welded or casted flanges in both ends. The size of the Surface Discharge Plate depends on the outer diameter of the tube well. For cable entry and ventilation of the well there should be 2 holes each rectangular holes suitable for the flat cable.

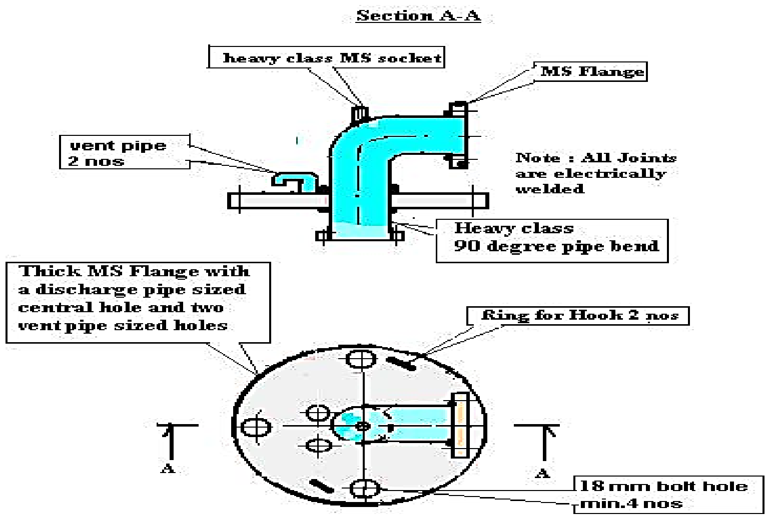


Figure 55: Surface Discharge Plate

c) **<u>Pressure gauge</u>**

P = 0-1 MPa with 20 mm outer threaded connection together with stop cock and other accessories for connecting to discharge bend of above surface plate

d) **<u>Check (non-return) valve</u>**

Generally cast iron double flange (CIDF) swing type check valve (non-return) confirming to IS: 780 Pn 1.6 Hydraulic Test Pressure Rating : (body test : 2.4 MPa , Seat test : 1.6 MPa) shall be used when the pumping head is less than 75 m.

e) **<u>Delivery (gate) Valve</u>**

Cast iron double flange (CIDF) gate valve confirming to IS: 780 Pn 1.6, Hydraulic test Pressure Rating: (body test: 2.4 MPa, Seat test: 1.6 MPa) shall be used when the pumping head is less than 75 m.

When pumping head is high, use cast steel (CS) gate valve confirming to BS 1414 or equivalent standard of required class.

f) **<u>Submersible Cable: IS: 694 (1990) or equivalent</u>**

Flat PVC insulated copper conductors. The conductor insulation shall be water and oil resistant suitable for continuous immersion.

It shall be sized to limit the voltage drop to 2.5% at the motor’s terminals with 15 m extended length.

Generally, submersible motor pump is supplied only with 3 m – 5 m length of submersible cable. Additional cable should be jointed as per requirement. On jointing the submersible cable care should be taken so that the cable joint should be sufficiently strong and waterproof. Either ***Cold Shrink*** or ***Hot Shrink*** jointing material of standard make should be used.

g) <u>Motor Control Panel</u>

Motor control panel must be safe to operate the pump motor. Provisions for the protection of motor against over loading, dry running, single phasing and opposite phasing should be provided.

It should have the following components:-

a) Molded case Circuit Breaker (MCCB) of suitable capacity - AT /AF = 1.5 x line current / 3 x line current

b) Fully automatic air break type Starter with magnetic contactors of capacity: 3 X line current for DOL and line current rating for Star-Delta, Bi-metallic overload relay (phase current ± 20%) and electronic timer.

Use DOL max up to 7.5 HP, STAR/ DELTA : 7.5 HP to 40 HP and Autotransformer/Soft Starter:

above 40 HP motor

c) Low Water Level Guard/ Float- less Switch

d) Single Phase Preventer and Phase Sequence Relay.

e) Voltmeter with Selector Switch between all phases.

f) C/T Ammeter with Selector Switch.

g) Push Button Switches for 'START' and 'STOP' the motor.

h) Indicating Lamps for various actions.

i) Cable connector (2 x full current capacity)

Note: Power voltage shall be 3 P 400 ±15% and control voltage shall be either 220 V or 400 V

j) Proper earthing should be provided to all electrical devices like, motor control panel, distribution box (DB), pump prime mover etc.

6.1.3.6 Pump Lifting Arrangement

An I- beam or a sufficient diameter MS/GI pipe beam supported on two RCC pillars at least 1 m higher than the roof slab should be provided directly over the well which should be capable to bear at least 3 times the weight of complete pump unit and water in the column (riser) pipe. A chain pulley block should be fitted to the beam.

6.1.3.7 Water Discharge (Delivery) Pipeline

Discharge pipeline delivers water either directly to the OHT or to water treatment plant. In some case provision of by-pass is made to transport water to both units with the operation of a control valve to apply additional head when there is a risk of over pumping the well. In such case it is better to provide an appropriate sized orifice in the low head side of the pipeline.

Diameter for most economical flow velocity should be selected. Flow velocity in the pumping main may be selected as v = 0.5 to 1.5 m/s. Lower velocity is for long pipeline and higher for short pipeline. The size of the discharge piping may be selected of one size higher than the nominal delivery size of the pump.

A check (non-return) valve is installed next to the surface discharge plate followed by a gate (discharge) valve. A washout valve is installed next to the discharge valve. Before the gate valve a pressure gauge is installed and in case of long riser pipe an air release valve is to be provided on the discharge bend of surface plate. In case of high flow rate and/ or long discharge pipeline arrangement for surge protection is necessary. A dismantling joint must be provided between the pump and the valves.

6.1.4 Subsurface Water Pumping Station

When a large water supply is needed and/or a shallow aquifer near a source of recharge (a lake or river) is available, a collector well is the first choice in practice A horizontal collector well consists of a vertical concrete caisson (shaft of concrete rings with a diameter of 2 – 4 m), and lateral well screens (usually named legs or laterals) extending radially from the caisson. A lateral is typically 50 m long (also lengths of more than 100 m are possible). There may be one to ten or more laterals at multiple elevations depending on the hydrogeological properties of the aquifer.

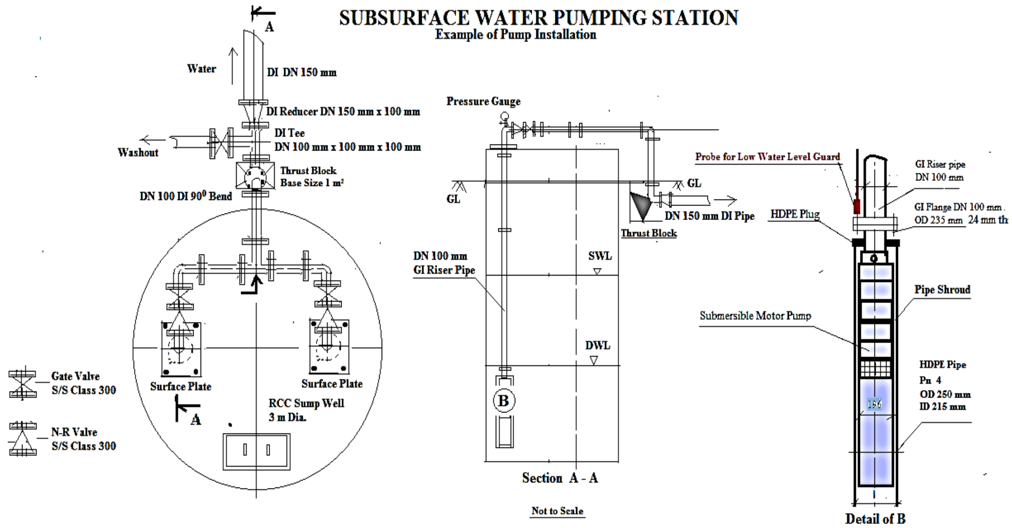


Figure 56: Sub-Surface Water Pumping Station

1. Big diameter, less stage deep well submersible pump or suitable vertical open well submersible pump may be used.
2. As the flow rate around the motor surface is not sufficient for cooling the motor a suitable pipe shroud is to be used as shown in above drawing.
3. Control valves may be installed on the top slab of the well but, not inside the well. Pump should be easily lifted out of the well from the top.

\

6.2 Type of Electric Power Supply and Electrical Control Devices

Types of electric power supply:

a) 11 KV (sometime 33 KV) 3 P High (Medium) Tension Power Supply

b) 400V/220 V TPN Low Tension Power Supply

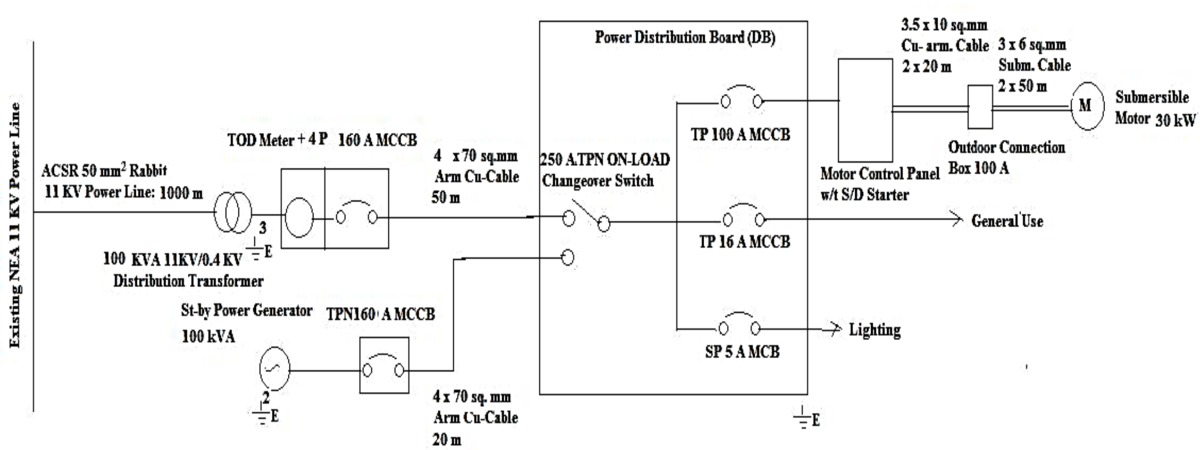


Figure 57: Typical Power Distribution Single Line Diagram

Generally, electrical component of a drinking water pumping station is consisting of followings:

i. 11 KV power tapping and transmission line
ii. 11KV/0.4 KV distribution transformer station
iii. 0.4KV power distribution system consisting of distribution board, motor control panel and pump station lighting system, and
iv. Standby power generation unit (Diesel Power Generator)

6.2.1 11 KV High (Medium) Tension Power Supply

On designing and cost estimating it is necessary to follow guideline of Nepal Electricity Authority (NEA).

6.2.1.1 Main components of 11 KV High (Medium) Tension Power Line

High tension power is transmitted from NEA power line through overhead line. Successful operation of an overhead line depends to a great extent upon the mechanical design of the line. Strength of the line should be such so as to provide against the worst probable weather conditions. Main components of an overhead line are:

a) Conductor
b) Supports
c) Insulators
d) Cross Arms
e) Miscellaneous Items

Conductor material generally used is Aluminum Conductor Steel Reinforced (ACSR). The ACSR conductor shall be fabricated in accordance with BS: 215/ IEC: 209, latest revision. Followings conductors are generally used:

Table 20: Type of Conductors Used

Code Name	Nom. area, mm2	Stranding (Al/Steel)	Breaking Strength kN	Mass, kg/km	Resistance at 20^0C (Ω·m)
Dog	100	6/7	32.7	394	0.273
Rabbit	50	6/1	18.35	214	0.540
Weasel	30	6/1	11.4	126	0.907

In many cases it is sufficient to use Weasel however, for more strength recommended to use Rabbit.

Line supports should have following properties:

i. High mechanical strength to withstand the weight of conductors and wind loads etc.
ii. Light in weight without the loss of mechanical strength.
iii. Cheap in cost and economical to maintain.
iv. Longer life.
v. Easy accessibility of conductors for maintenance.

Following line supports in 11 m length are used:

a) Pre-stressed (PSC) pole confirming to Indian Standard No. IS: 1676-1978 of latest revision.
b) Galvanized Steel Tubular pole confirming to BS 6323 or Indian Standard No. IS: 2713 (Parts I to III)-1980 of latest revision.
c) Galvanized Telescopic Tubular pole (Ranger Mast Pole) fulfilling requirements of BS 6323 Part 1 to 8 steel tubes. The telescopic pole sections and fittings shall be manufactured from standard steel as per BS 4360 Grades 43 C, D, E or 50 C, D, E or equivalent national/ international standards.

Insulators provide necessary insulation between line conductors and supports and thus prevents any leakage current from conductors to earth. Three types of insulator namely, i) Pin Insulator, ii) Disc Insulator and, iii) Stay Insulator are used. Pin and Disc insulators shall be manufactured and tested in accordance with IS: 731-1971 and IS: 3188 however, stay insulator shall be manufactured and tested in accordance with IS: 5300-1969.

Cross arms provide supports to the insulators. The steel cross-arms shall be fabricated from hot-rolled channels and angles. The steel channels and angles shall be fabricated and tested in accordance with Indian Standards IS: 226-1975 and IS-808-1964 or any revision thereof or other equivalent national or international standard provided that ensure at least equal or better quality to the standard mentioned above. The minimum tensile strength of the steel shall be 4200 kg/cm2.

The conductor sag, the difference in level between points of supports and the lowest point on the conductor should be kept to a minimum in order to reduce the conductor material required and to avoid extra pole height for sufficient clearance above ground level. It is also desirable that tension in the conductor should be low to avoid the mechanical failure of conductor and to permit the use of less strong supports. In an overhead line sag should be so adjusted that tension in the conductors is within the safe limits. The tension is governed by conductor weight, effect of wind and temperature variation. In high altitude ice load during winter shall also be considered. It is a standard practice to keep conductor tension less than 50% of its ultimate tensile strength i.e. minimum factor of safety in respect of conductor tension should be 2.

6.2.1.2 Transformer Substation

A substation is used to step down 11 KV high voltage (Tapped from NEA power line) to 400 V/220V in which pumping equipment and their control devices are operated and also the pumping station is lighted. A typical substation includes:

i. Distribution transformer of required capacity.
ii. Disconnecting switch gear.
iii. Lightning arrestor, and
iv. Earthing arrangement.

Generally, smaller transformers are pole mounted and higher capacity transformers are installed on suitable elevated platforms.

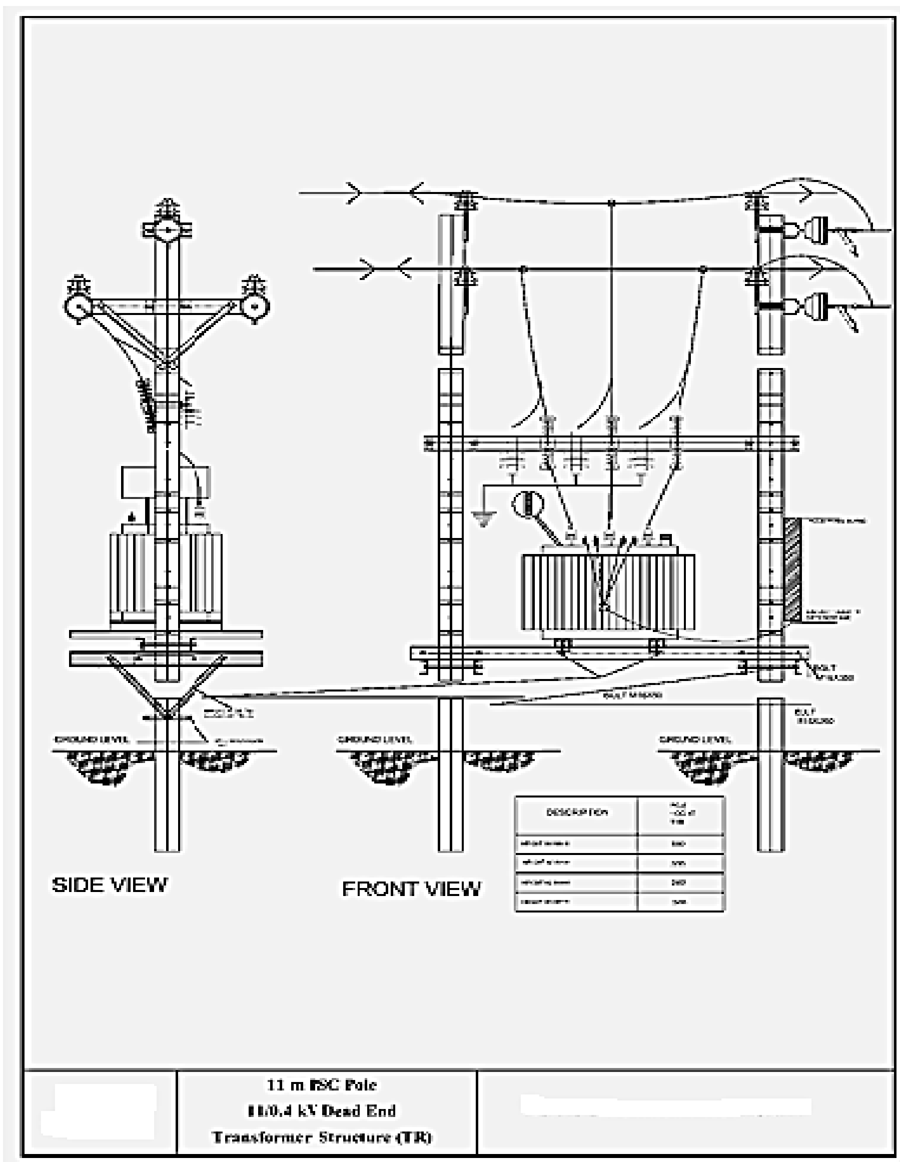


Figure 58: Typical Transformer Station

Table 21: Required Accessories for Transformer Station Assembly

S.No.	Quantity	Units	Material
1.	6	Nos	Pin Insulator with Pin and Nuts/Washer
2.	2	Nos	Steel Cross Arm Channel (50x100x6, 4 x 300)mm
3.	2	Nos	Pole Clamp with Nuts. Bolts and Washers (PC1)
4.	2	Nos	Steel Cross Arm Channel (500x100x6, 4 x 1200)mm
5.	2	Nos	Pole Clamp with Nuts, Bolts and Washers (PC2)
6.	4	Nos	Flat Cross Arm
7.	3	Nos	9 kV Surge Arrestor
8.	3	Nos	Distribution Cutout with Fuse Holders
9.	1	Nos	Channel For LA & DO ISLC 100 2348 mm
10.	2	Nos	Platform Channel; (TR1) ISMC 100 2500 mm
11.	2	Nos	Platform Channel (TR3) ISMC 100 2500 mm
12.	4	Nos	Platform Channel (TR2) ISMC 100 1200 mm
13.	2	Nos	Platform Channel (TR4) ISMC 100 1200 mm
14.	8	Nos	Bracing Angle (TR5) 50 x 50 x 50 x 1 mm
15.	2	Set	Bracing Band (TR6 or TR6P) with 2-M16 Bolt, 2-M16 x 50 Bolt, 8-M16 Nut, 8-M16 Washer
16.	16	Nos	M16 x 50 Bolt with 2-M16 Nut, 2-M16 Washer
17.	8-STTP 12PSC	Nos	M16 x 250 Bolt with 2-M16 Nut, 2-M16 Washer
18.	8-STTP 12PSC	Nos	M16 x 350 Bolt with 2-M16 Nut, 2-M16 Washer
19.	1	Nos	Transformer
20.	3	Nos	Transformer Earthing
21.	As Req.	M	Grounding Conductor (Copper)
22.	6	Nos	Preform Ties
23.	2	Nos	Steel Tubular Pole / PSC Pole

Sizing of distribution transformer is done according to the maximum possible power load in the pumping station. It is sized in such a way that the maximum load on it does not exceed 65% of its nominal power rating.

Distribution transformer should be outdoor type, copper winded manufactured by NS and ISO certificates holder as per IEC specifications and/or other recognized international standards. It should be tested in a laboratory recognized by Nepal Electricity Authority (NEA).

Individual earthing should be provided for lightning arrestor, transformer neutral and transformer body. Any of standard plate, pipe or chemical earthing approved by NEA may be provided.

6.2.2 400V/220 V Low Tension Power Distribution System

After step down to 400 V/220 V electricity is conveyed through three phase neutral (TPN) line, first to NEA kilowatt hour meter and cutout switch consisting of suitable capacity four pole molded case circuit breaker (MCCB). It is recommended to use Time of Day (TOD) meter. These shall be fitted in an outdoor/ indoor type, totally enclosed, metal clad front standing and dust and vermin proof Main Switch Board.

Electric power should be distributed to different part of pumping station by a main distribution board (MDB). MDB should consisting one Main MCCB/ on-load changeover switch and calculated capacity MCCB for each device present in the pumping station. MCCB should be sized not exceeding to 1. 4 times the full load current to be supplied. The MCCB rated 50 Amperes through 200 Amperes shall be furnished with thermal-magnetic or static trip. The MCCB rated 250 - 500 Amperes shall be furnished with Thermal-adjustable magnetic or static trip. Provision for general use and future extension shall also be made. Busbars shall be of high conductivity copper bar of sufficient cross-sectional area so that a current density of 325 amp.Per sq.cm is not exceeded at normal current rating and supported on non-hygroscopic insulator.

The neutral busbar cross section shall be not less than 50% of the phase busbars.

3.5 core or 4 core armored or unarmored copper cable should be used with sufficient size limiting voltage drop under 2.5 % at the end point.

6.2.2.1 Cable Laid Underground

Where cables are laid underground they shall be laid in a trench to a depth of 70cm [minimum] from the ground level. Care shall be taken to avoid interference with underground structures i.e. water pipes, sewerage lines etc. Any telephone lines or other cables coming on the way shall be properly shielded as directed by the site engineer. After the excavation of the trench to a specified depth and the width of the trench governed by the no of cables to be buried and the convenience of the digging the cable/cables shall be laid at the bottom of the trench. The bricks on edges shall be laid along the cable on either sides. The brick canal so formed shall be filled with chemically inert sand and top of the bricks on edges shall be bridged across by the brick. The completed brick structure shall looks like a sand filled inverted brick canal. The road crossing shall be avoided as far as practicable. The ductile iron pipe protection for the cable shall replace the brick protection across the road crossing. After cable pulling through the ductile iron conduit they shall be plugged on either end.

6.2.2.2 Cables Run Over Horizontal or Vertical Surface

Wherever cables are to run along wall surface of either the building or electrical duct or on the ceiling, these shall be fixed with cleats. Cleats shall consist of molded insulated materials divided in two halves and secured to suitable racks made of angle iron or flat steel of suitable approved section. The securing shall be by means of studs and nuts with locknuts and washers. For PVC armored cables, aluminum or G.I. Claw type Clamps may be used.

6.2.2.3 Passing Through Walls And Floors

Where conductors pass through walls, one of the following methods shall be employed. Care shall be taken to see that wire pass freely through protective pipe or box and that wire pass through in a straight line without any twist or cross in wires on either ends of such holes.

a) A metal box extending through the whole thickness of the wall and casings or conductors shall be carried so as to allow 1.3 cm air space on three sides of the casing or conductor.

b) The conductor shall be carried either in a rigid steel conduit conforming to accepted standards or a rigid or semi-rigid non-metallic conduit conforming to accepted standards.

6.2.2.4 Passing through Walls and Floors

Where conductors pass through walls, one of the following methods shall be employed. Care shall be taken to see that wire pass freely through protective pipe or box and that wire pass through in a straight line without any twist or cross in wires on either ends of such holes.

a) A metal box extending through the whole thickness of the wall and casings or conductors shall be carried so as to allow 1.3 cm air space on three sides of the casing or conductor.

b) The conductor shall be carried either in a rigid steel conduit conforming to accepted standards or a rigid or semi-rigid non-metallic conduit conforming to accepted standards.

6.2.2.5 Overhead Distribution Line

The low voltage overhead distribution line shall be installed over pre-stressed concrete pole / steel tubular poles. The sizes of the supports to be used depend according to span, number of conductors and sizes of the conductors to be carried. The conductors shall be mounted on shackle insulators fixed to the poles by means of D-iron and bolts.

Suitable straps and shackle insulator shall be used for teeing. The arrangement of conductors in vertical plane from top to bottom shall be in the following order.

First conductor - A phase

Second conductor - B phase

Third conductor - C phase

Fourth conductor – Neutral

6.3 Diesel Power Generator

6.3.1 Auxiliary Power Generator Capacity

An auxiliary power generator can be installed as a safe power source in case of service interruption. The generator capacity should be calculated to satisfy the requirements of loads that have been determined, taking into account the operation of the pump installation in combination with any devices that must be operated at the same time together with total lighting power, control power and other loads operating at the same time. However the capacity of the auxiliary power generator shall be limited to the minimum as required.

The capacity of generator must be calculated with the following considerations:

- Normal load capacity
- Starting load capacity
- Starting voltage drop

An induction motor is loaded maximum during its start and voltage drop is also maximum during the start. Hence, it is recommended to select capacity of generator for starting load capacity (i.e. for direct start 6Q and for start-delta start 2Q when Q is normal load capacity).

Following table may be used for generator sizing:

Table 22: Generator Sizing

Electric Motor Size		Minimum Generator Required (KVA) by Starting Method		Material
HP	**KW**	**D.O.L to KVA Size (Note 1)**	**S.D to KVA Size (Note 2)**	**KVA Used When Running (Note 3)**
1	0.75	2.5	2	1
1.5	1.1	3.75	3	1.5
2	1.5	5	4	2
3	2.2	7.5	6	3
4	3	10	8	4
5	3.7	12.5	10	5
6	4.5	15	12	6
7.5	5.5	18.75	15	7.5

Electric Motor Size		Minimum Generator Required (KVA) by Starting Method		Material
HP	KW	D.O.L to KVA Size (Note 1)	S.D to KVA Size (Note 2)	KVA Used When Running (Note 3)
10	7.5	25	20	10
12.5	9.3	31.25	25	12.5
15	11	37.5	30	15
20	15	50	40	20
25	19.6	60.5	50	25
30	22	75	60	30
40	30	100	80	40
50	37	125	100	50
60	45	150	120	60
75	55	187.5	150	75
100	75	250	200	100
125	90	312.5	250	125
150	110	375	300	150
175	130	437.5	350	175
200	150	500	400	200
250	185	625	500	250
300	225	750	600	300
400	300	1000	800	400

Size the generator to run between 60-80% full load rating of generator

Diesel Generator Set complete with engine, alternator, microprocessor based control system and acoustic canopy mounted on a common base frame with AVM pads, in built fuel tank of suitable capacity for one day's operation, residential silencer with exhaust piping, etc. complete conforming to IS0 8528 specifications and CPCB certified for emissions and raise compliance should be selected.

6.3.2 Installation of a Diesel Generator Set

6.3.2.1 Location

Dust and Fumes are the greatest danger to generating set as they could lead to clogging of cooling and air system which may affect Genset performance.

1. If Genset is to be installed in Free field condition, ensure dust free location and clear space of 2 times the height of Genset.
2. If Genset is to be installed in the room, ensure proper cross ventilation, clear space of minimum.3 meters from all sides.

3. If Genset is to be installed on a roof top, ensure structural clearance, proper cross ventilation with respect to wind direction, clear space of minimum 3 meters from all sides.
4. If Genset is to be installed in the basement, ensure ventilation with respect to Air requirement and clear space for easy maintenance.

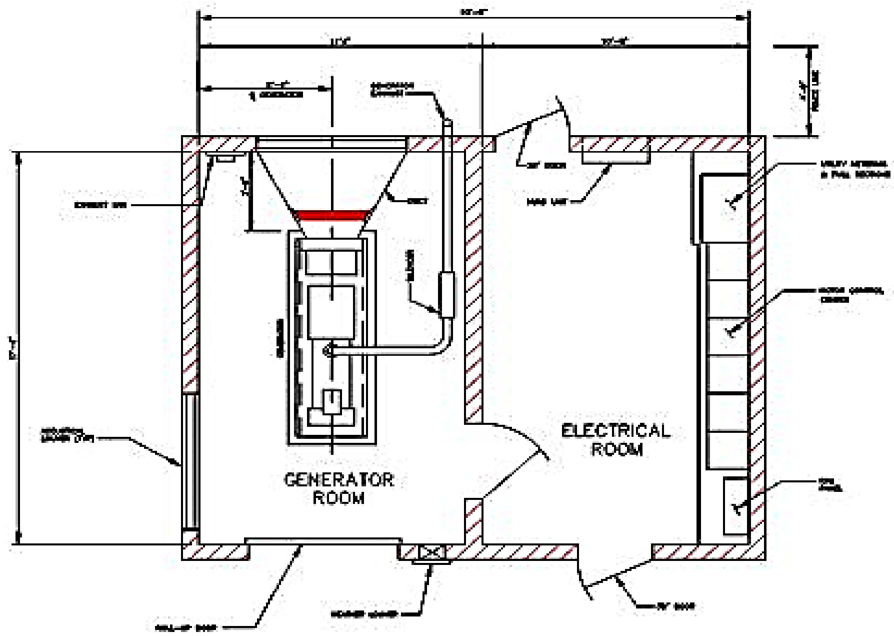


Figure 59: Typical Plan for Layout of Diesel Generator

6.3.2.2 Foundation

The length and width of foundation should be at least 300 mm more on each side than acoustic enclosure length and width respectively.

It is recommended to have foundation height of 150 to 200 mm above ground level to maintain cleanliness and avoid flooding.

Check the foundation level diagonally as well as across the length and width for even flatness and same should be within + 50 mm in horizontal plane in any one direction only.

Ensure that the concrete is completely cured before positioning the enclosure.

Ensure that the foundation to support 1.5 times of the total wet weight of the single generating set installation and 2 times of the total wet weight for the multiple generating set installation.

For rooftop installations, it is necessary to have planning and structural design approval.
Do not install acoustic enclosure on loose sand or clay. Avoid hard / sharp projections such as stone or steel parts on foundation surface, which may damage fuel tank at the bottom.

Avoid uneven foundation which may lead to improper resting of Genset on foundation, there by leading to leakage of sound and vibration on Genset.

If Genset is to be mounted on foundation having uneven surface under unavoidable circumstances, use appropriate thickness of rubber matting above foundation for positive sealing of Genset with base.

6.3.2.3 Earthing

Any leakages in current will be earthed through the shortest route in the link.

Genset should be connected to earth in accordance with the instruction given by the manufacturer.

The size of the link used for the main earth connection should be of adequate size.

The generating set and all associated equipment must be earthed before the set is put into operation.

4 numbers of earth pits are required

- 2 earthing pits for Genset / control panel body
- 2 earthing pits for neutral.

Resistance between 2 earth pits should not be more than 5 Ω.

6.3.2.4 Access

Personnel access should be convenient for routine inspections. Access for installation and replacement of the unit should be taken in consideration. Future equipment installation or construction should not hinder the access for the replacement of parts or the complete diesel generator unit.

6.3.2.5 Maintenance

Adequate space should be provided for normal maintenance and overhaul of the diesel generator set as shown in Figure 52.

6.3.2.6 Noise

A location away from principal working areas or offices and typically a generator set comes with the calculate dBA levels at full load operation. This is the honest way to report the noise level. Vibration of a diesel generator set should be taken in consideration and kept away from quiet working location.

6.3.2.7 Ventilation

The room or space where the generator operates should not exceed 100 degrees. Generators installations require an intake of cool, clean air and an outlet vent for hot air. The size of the space affects room temperature. A Generator in a small space will affect the room temperature and may require extra ducting. Increasing the vent sizes may cool the room down and ensure a positive airflow. Moist air is corrosive to a generator set, so make sure inlets are positioned to minimize moisture intake. High humidity can be damaging to a diesel generator set.

6.3.2.8 Exhaust Piping

The location should provide a short, direct route to an outdoor termination which will not exhaust directly on personnel or near an intake vent.

6.3.2.9 Fuel Piping

Extreme care should be taken when designing and installing the fuel system. Fuel lines should have as few connections as possible and kept away from hot engine or exhaust components. The fuel tanks should be level with or below the set to prevent siphoning in the event of a line failure.

6.4 Domestic Water Meters

6.4.1 Water Meter and its Constituents

6.4.1.1 Water Meter

A water meter is an instrument intended to measure continuously, memorize and display the volume of water passing through the measurement transducer at metering conditions. It included at least a measurement transducer, a calculator (including adjustment or correction devices if present) and an indicating device. These three devices may be in different housing).

6.4.1.2 Measurement Transducer

A part of meter which transforms the flow or the volume of water to be measured into signals which are passed to the calculator. It can be based on a mechanical or an electrical or an electronic principle. It may be autonomous or use an external power source. The measurement transducer includes the flow sensor or volume sensor.

6.4.1.3 Flow Sensor or Volume Sensor

That part of water meter (such as a disc, piston, wheel turbine element m or electromagnetic coil) which senses the flow rate or volume of water passing through the meter.

6.4.1.4 Calculator

A part of the water meter which receives the output signals from the transducer(s) and possibly, from associated measuring instruments, transforms them and stores the results in memory.

6.4.1.5 Indicating device

A part of water meter which displays the measurement results either continuously or on demand.

Other devices like, adjustment device, correction device, ancillary devices consisting of : zero setting device; price indicating device; printing device ; memory device tariff control device etc. may be incorporated in the meter.

6.4.2 Objectives of metering

Metering of water supply is usually motivated by one or several of four objectives:

- It provides an incentive to conserve water which protects water resources (environmental objective)
- It can postpone costly system expansion and saves energy and chemical costs (economical objective)
- It allows a utility to better locate distribution losses, and prevents backflow (technical objectives)
- It makes possible to charge customers in proportion to the water they use. It is fair for all customers. It allows the system to demonstrate accountability. (Social objective)

Metering is considered good practice in water supply only when followings are correctly done:

- Selection of appropriate meter
- Proper installation of meter
- Maintenance and calibration of meter
- Reporting of meter readings
- Random verification of meter accuracy

6.4.3 Common Types of Meter

There are several types of meter available on the market which are classified into two basic types:

a) Volumetric (Positive Displacement) Meters
b) Velocity meters

In volumetric type of meter, a known volume of liquid in a tiny compartment moves with the flow of water and is operated by repeatedly filling and emptying these compartments. The flow rate is calculated based on the number of times these compartments are filled and emptied. The movement of disc or piston drives an arrangement of gears that registers and records the volume of liquid exiting the meter. There are two types of positive displacement meters: nutating disc and piston among which the piston type of meters are commonly used in Nepal.

Volumetric Type **Combination Type** **Velocity Type**

Figure 60: Common Types of Flow Meters

Velocity meters operate on the principle that water passing through a known cross-sectional area with a measured velocity can be equated into a volume of flow. Velocity meters come in different types, including turbine, single and multi-jet, propeller, ultrasonic, venture, and orifice meters.

Volumetric meters are good for low flow, have accuracy up to class C. Velocity meters are used for high flow.

Compound meter is combination of both a positive displacement and velocity meter installed together-to be able to measure high and low flows with a single meter. Low flows are measured through positive displacement while high flows are measured by velocity. A valve arrangement directs flows into each part of the meter.

6.4.4 Selection of Meter

Meter should provide adequate accuracy, should be easy to install and maintain, and inexpensive. It is selected using several factors: flow rate, size of pipe, pressure loss etc. However, the first consideration in selecting a meter is whether or not it meets the requirements of following metering order:

- Meter shall be a totalizing flow meter (keeps a cumulative flow count)
- Meter shall have sufficient digits so that "roll over" to zero does not occur within a three year period.
- Meter must have a factory – rated accuracy of at least class C

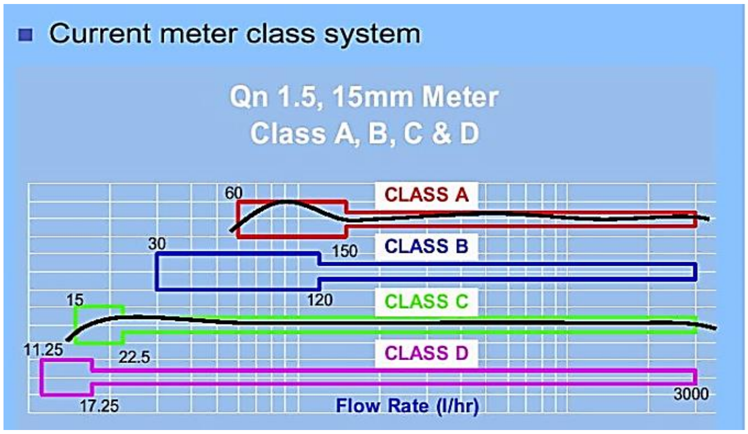


Figure 61: Current Meter Class System

Table 23: Class of Current Meters and their Discharge

Nominal Diameter (mm)	Q_n (m^3/hr)	Q_{max} (m^3/hr)	Class A		Class B		Class C		Class D	
			Q_{min} (l/hr)	Q_t (l/hr)	Q_{min} (l/hr)	Q_t (l/hr)	Q_{min} (l/hr)	Q_t (l/hr)	Q_{min} (l/hr)	Q_t (l/hr)
15	1.5	3.0	60	150	30	120	15	11.25	22.5	17.25

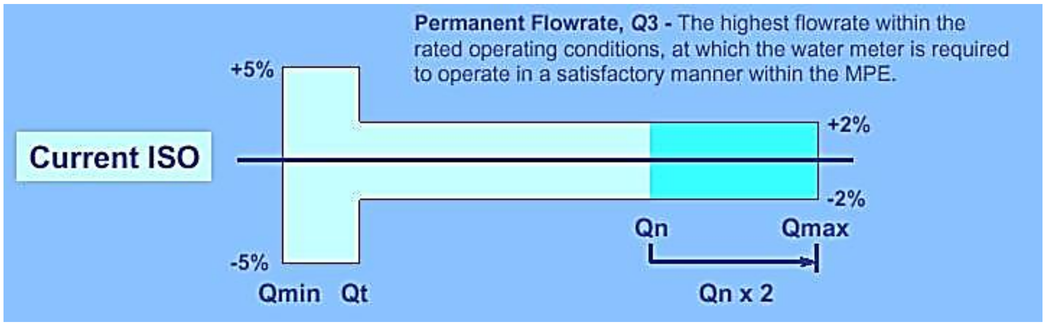


Figure 62: Permanent Flowrate

Where, Qt is transitional flow rate at which the maximum permitted error of the meter changes from ± 5% to ± 2%

- Meter register cannot be reset

- Meter shall have waterproof and tamperproof seal.
- The units of measurement and multiplier shall be displaced on the meter.

To assist in the selection of appropriate meter, it is useful to develop a list of meters known to meet the requirements. If a private tap owner chooses to install a new meter or a meter supplier wants to introduce a new meter that is not on the pre-approved list, documentation (such as manufacturer literature) together with sample meters must be provided showing that the meter meets the requirements of the order. After investigation and testing of the sample meters the new meter may be accepted and be included in the list if it meets the requirements of order.

6.4.5 Meter Sizing

Water meters are sized on their nominal flow rate. This is called the Qn and is given in cubic meters per hour (one cubic meter is 1,000 liters of water). The water meters maximum flow rate is twice the Qn.

1. If the required flow rate is known then a water meter can be selected so that the required flow rate falls between the nominal and maximum flow rates.
2. If the flow rate is not known then it is generally safe to select a meter of the same nominal size (DN) as the pipework it is to be connected to.

Oversized water meters not only increase the capital investment but also considerably reduce the measuring accuracy during periods of low water flow. Undersized water meters can easily be overloaded which may cause premature wearing of parts.

6.4.6 Class of Meter

The class does **not** indicate the accuracy of the water meter but at what flow rate the meter meets the common accuracy figures. These are ± 5% at the meters minimum flow rate and ± 2% in the meters normal range (between Qt and Qmax) for cold water meters. The figures for hot water meters are ± 6% and ± 3% respectively. The higher the class of water meter the higher the accuracy at very low flow rates; Class D having the highest accuracy, and class A the lowest.

When deciding if a low flow reading is required it should be remembered that even a class A Qn 2.5 (a 3/4 " meter) will start to read, within its tolerance band, at a flow rate of 1.66 l/m (a basin tap will flow at between 6 and 10 l/m)

6.4.7 Wet, Liquid sealed or Dry dial

Wet dial meters are used for cold water applications where the meter is subject to climactic changes (e.g. a meter mounted outside a building but still protected from frost) which could cause condensation to form on the face of the dry dial meter making it difficult to read.

Liquid sealed dial can keep long term service to read clearly. This should be balanced against the possibility of water borne contamination getting into the meter. The type of meter must be selected based on site conditions, but in all cases dry dial meters should be used in applications where the water quality is suspect, i.e. Contaminated or cloudy.

6.4.8 General Technical Specification of Domestic Water Meter

NS 428-2058 ISO 4064 (part 1, 2 and 3. (With some inclusion of IS: 977 where it permits)

TECHNICAL SPECIFICATION OF DOMESTIC WATER METER

1. Type

Volumetric rotary piston type in-line water meter to be fitted directly in a closed conduit by means of the threaded end connections provided, for the measurement of potable water. In general it shall be confirmed to ISO 4064 or equivalent.

2. Size

Nominal size of water meter shall be 15 mm. (½ in).

3. Materials

The water meter shall be manufactured from materials of adequate strength and durability that shall not be adversely affected by the water temperature variations, within the working temperature range (min.0.10 C to max.500 C). All parts of the water meter in contact with the water flowing through it shall me manufactured from non-toxic, non-contaminating and biologically inert. The complete water meter shall be manufactured from materials that are resistant to internal and external corrosion, or that are protected by a suitable surface treatment.

Unless and otherwise not specified the body of the water meter shall be made from bronze.

Nominal working pressure of the water meter shall be 10 kg/cm2 and the test pressure shall be 20.6 bar (21 kg/cm2)

4. Design and Construction

Water meter shall be simple in construction – enable speedy replacement of all internal components. It shall ensure maximum accuracy at all flow rates. It shall be consist of tamper proof, liquid filled sealed counter unit ensuring good readability under all conditions. Measuring chamber of water meter, the registration box, the cap and the lid shall be housed in the meter body. It shall not contain separate gear train.

Registration shall be by direct reading with digital counter registering in liters and cubic meters (m3). The color black shall preferably be used to indicate the cubic meter and its multiples and the color red shall be used to indicate sub-multiples of a cubic meter in the digital counter. Minimum readable quantity shall be 1 liter and maximum readable quantity shall be 9,999.99 m3 and resets to zero at 10000 m3. Lowest flow rate at which the water meter shall start flow registration (Q1) shall be 10 liters per hour and permanent flow rate – the highest continuous flow rate (Q3) at which the water meter is required to operate in a satisfactory manner within the maximum permissible error shall be 2700 liter per hour. The pressure loss of the meter shall not exceed 0.063 MPa (0.63 bar) at any flow rate between Q1 and Q3 inclusive.

Pressure difference across the sealed counter unit shall be eliminated by means of an automatic compensating device.

Water meter shall have an inbuilt non-return valve on the outlet end to prevent back flow. It shall have watertight dismantling coupling on each side to allow easy removal of the unit from the pipe work. Sealing holes shall be provided and meter shall be sealed in such a manner as to render it impossible to obtain access to the measuring unit including registration box. The sealing wire shall be rust proof.

Overall length of the meter excluding connectors shall be less than 165 mm and including both connectors shall not exceed 220 mm. Overall height and width (or diameter) shall be less than 100 mm. Net weight of the water meter shall be nom.1 kg without connector and total weight including connectors shall be around 1.2 kg.

5. Metering Accuracy

Water meter shall provide maximum accuracy and reliability when mounted in horizontal, vertical or in any position. Maximum permissible errors during flow rate test at maximum admissible working pressure (MAP) shall be within ± 2 % from minimum to permanent flow rate and within ± 5 % from permanent to overload flow rate - highest flow rate at which the meter is required to operate for a short period of time within its rated operating conditions.

6. Tests

Water meters shall be tested by a recognized testing authority (may be the manufacturer) according to ISO 4064 –3 and the test certificate including general tests, static pressure tests, pressure-loss test durability tests and other performance tests shall be submitted along with randomly selected sample meters (as mentioned in table below) before the delivery of each lot. These Sample meters shall be subjected to further tests as per IS 779 - 1978 in presence of representative of the project office consisting of hydrostatic test, flow test including loss of head at minimum to permanent flow rate and metering accuracy.

Table 24: Sample Size and Criteria for Acceptance

Size of the lot	Size of first sample	Acceptance Number (a_1)	Rejection Number (r_1)	Size of Second Sample (if required)	Size of cumulative Sample	Cumulative Acceptance Number (a_2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Up to 50	5	0	2	5	10	1
51 – 150	8	0	2	8	16	1
151 – 300	13	0	3	13	26	3
301 – 500	20	1	4	20	40	4
501 – 1000	32	2	5	32	64	6
1001 and above	50	3	7	50	100	8

Criteria for acceptance: If in the first sample the number of defective meters is less than or equal to the corresponding acceptance number as given in col.3 of Table the lot shall be declared as passing the acceptance tests. If the number of defective meters is greater than or equal to the corresponding rejection number r1 given in col.4 of Table, the lot shall be declared as not passing the acceptance tests. If the number of defectives is greater than the acceptance number a1 but less than the rejection number r1 the second sample of size equivalent to that of the first shall be taken and subjected to acceptance tests. The number of defective meters found in the first and the second sample shall be added and if the cumulative number of defectives thus obtained is less than or equal to the acceptance number a2 given in col. 7 of Table , the lot shall be declared as passing the acceptance tests, otherwise it shall be rejected.

However each water meter before installation in the consumer’s tap must pass the accuracy and other hydraulic tests.

The cost of testing should be included in the cost of meter. No additional payment shall be made for testing of meter.

7. Marking

Each meter shall be permanently marked (embossed on meter body) with following information:

i. Manufacturer's Name or Trade Mark
ii. Model No.
iii. Serial number
iv. Nominal size of meter
v. Direction of flow of water on both sides of the meter.

8. Guarantee

Each water meter shall be guaranteed by the supplier against the defects in material, workmanship and performance under normal use for a period of at least 18 months from the delivery date.

9. Tools, Spare Parts and Accessories

- 1 set of tools for disassembling and reassembling the water meter shall be provided with each 500 nos of meter.
- 1 set of spare part including liquid filled sealed counter unit, measuring chamber and the registration box of water meter i.e. all plastics part shall be provided with each 25 nos of meter.
- 1 sealing pliers shall be provided with each 100 nos. of meter.
- 1 set of meter seal and sealing wire shall be provided with each meter.

6.4.9 Meter Installation

Installation Requirements

- A water meter should be installed in a location that is easily accessible, convenient for readouts, protected against flood and external effects.
- A water meter should have valves on both the inlet and the outlet that cut off water inflow so that the water meter can be removed for repair or maintenance. These valves should always be fully opened during the normal operation of the meter.
- The pipeline section that a water meter is mounted on should be shaped in such a way as to prevent the possibility of air entering inside the meter. The water meter should always be completely filled with water. Therefore the pipeline after the outlet of the meter should not descend below the level of the meter. Pipes bypassing the water meter are not allowed.

If there is a risk of air entering the meter, an upstream air release valve shall be incorporated and installed in accordance with the manufacturer's instructions.

- Gate valves, non-return valves, pressure reducing valves, and other fittings or incorrectly sized seals, which are installed in front of meters, create turbulence that can have a detrimental effect on meter accuracy. These fittings should be installed behind the meter.

It is a common accepted rule of thumb that straight sections of pipe of the same diameter, D, as the water meter , having lengths of 10D and 5D upstream and downstream of the water meter , respectively, are required and sufficient. It should be clarified that this is just a practical compromise. The longer the pipe, the better, particularly in the upstream side of water meter.

- Water should flow through a meter in the direction indicated by arrows put on the side(s) of the body. No backflow should be allowed. If no check valve is associated with the water meter, individual check valve should be installed in the downstream of the flow direction.
- Pipelines must be flushed and cleaned to remove sand, gravel prior to installing the meter. If impurities exist in normal water supply, the water meter should be fitted with a filter or dirt box on the upstream end of the meter.
- Water meters shall have protective devices installed, which can be sealed in such a way that after sealing and when the water meter has been correctly installed , there is no possibility of dismantling , altering or removing the water meter or its adjustment device without visibly damaging the protective devices.

6.4.10 Operation and Maintenance of Domestic Water Meter

After installation, water shall be let into the main slowly and with air bleeds opened so that trapped air does not cause the water meter to over speed, thereby causing damage.

A meter must be operated within its proper capacity range. It can be operated up to their full rated capacity without damage. However, continuous operation above 50% of maximum flow capacity should be avoided to prevent reduction in normal service life.

The maintenance of volumetric water meters is in general so simple that little attention has been paid to it by many users. The accuracy of water meter decreases over time. This accuracy decrease is accelerated in the presence of aggressive effect of water, particularly if it leaves deposits (iron, magnesia) causing premature wear of mechanical parts of water meter. Therefore, every single water meter should - after a reasonable period of operation - be removed and inspected or tested if necessary.

Accuracy should be checked prior to dismantling and cleaning. Chemical compounds that have a harmful effect on materials that water meter parts are made of should not be used. In particular aliphatic hydrocarbons such as petrol, xylene, toluene and some of their derivatives (i.e. acetone) must in no circumstance be used. If parts do need to be replaced, only original manufacturer spare parts should be used.

6.4.10.1 Operating Difficulties

1. **Register Stops**

   Clogging by dirt is common cause of stoppage and usually requires only a careful cleaning.

   Sometimes screw of gears comes loose, and Needs only to retightened.

2. **Runs inaccurately by a fairly constant percentage**

   If the error is not more than 5% and the meter is not too old, this may indicate either an error of original calibration, or an error resulting from slight wear.

   Once an error due to wear has started to develop, the meter may show further error within a relatively short time and should be watched.

3. **Over-registers erratically**

   Air is being passed through the meter along the water. The remedy is to keep the air out at its source, or in certain cases, to install an air release valve ahead of the meter. Scale formed inside a water meter also cause over-registration

4. **Under-registers by a substantial or erratic amount**

   This may be caused by severe wear or by partial clogging of meter by foreign matter. Open the meter and clean inside to correct the trouble, or if it needs further repair can be done locally if repair facilities and spare parts are available or send to factory for repairs.

5. **Dial Hand Travels Roughly**

   If the dial hand alternately stops and jumps ahead, this indicates difficulty with the meshing of the gearing, usually in the change gears. The gears may be worn to the point that the tops of the teeth strike on each other, or the gear may be adjusted so they mesh too tightly. In either case, adjust the change gears so they mesh smoothly with just enough clearance to prevent binding, and replace gears if worn.

6. **Leakage**

   Leakage around the meter body indicates usually that the operating pressure is above that for which the gasket was intended. Or the temperature is too high for the gasket causing it to burn out, or the meter was not correctly assembled after repair

6.4.11 TESTING OF DOMESTIC WATER METERS

Domestic water meters are tested according to ISO: 4064-3 and or IS: 6784-1984. Tests classified into three groups:

(a) Production routine tests
(b) Type tests, and
(c) Acceptance tests

6.4.11.1 Production Routine Tests

These tests shall be conducted on each and every meter after completion at the works. It shall consist of:

(a) Pressure tightness
(b) Loss of pressure
(c) Metering accuracy
(d) Minimum starting flow

6.4.11.2 Type Tests

These tests are necessary to check the performance and characteristics of the meter and its components and shall be carried out by a recognized testing authority (may be the manufacturer, if approved by the purchaser). The type tests shall comprise and be carried out in the following order:

(a) Pressure tightness test
(b) Flow test consisting of pressure loss, metering accuracy, minimum starting flow and temperature suitably tests.
(c) Construction
(d) Life test

6.4.11.3 Acceptance Tests

If the purchaser desires any of the production routine test to be repeated at the time of purchase, then, when agreed between the purchaser and the manufacturer. These tests may be carried at the manufacturer's works or at the place specified by the purchaser, provided that all arrangements for the test are made by the purchaser at the specified place.

6.4.11.4 Samples for Tests

Three water meters of same size, and class shall be sent along with six copies of manufacturer's detailed specification with figures for loss of head and accuracy curves, to a recognized testing authority for the purpose of type tests.

6.4.12 Equipment for Testing of Domestic Water Meters

The main equipment required for testing is as follows:

(a) Centrifugal pump and / or overhead tank
(b) Pressure gauge
(c) Collecting tank with position indicator
(d) High pressure and flow adjusting valves, and
(e) Stop watch or electronic timer

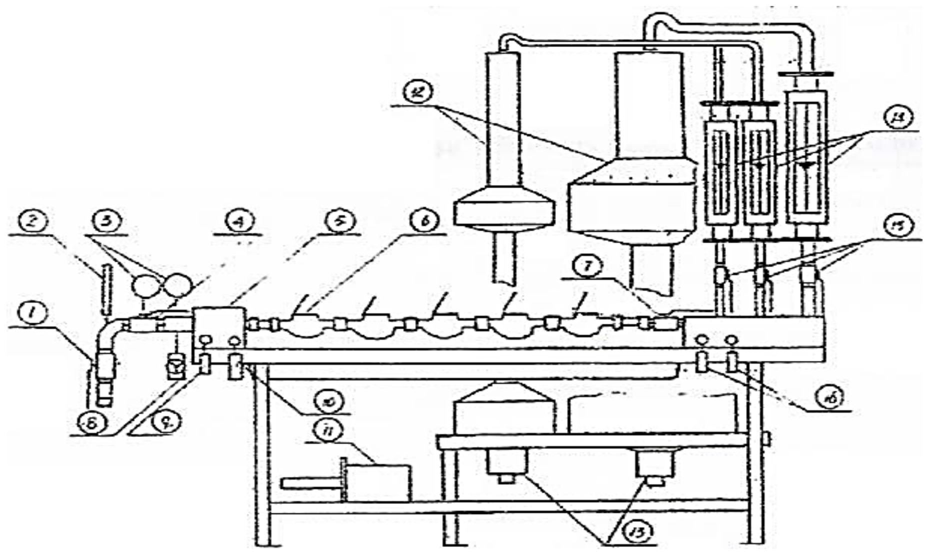


Figure 63: Water Meter Test Bench – (15-50mm)

1. Inlet Valve	2. Thermometer	3. Pressure Meter
4. Front high pressure valve	5. Clamper	6. Water meter
7. Back high pressure valve	8. Pressure increase valve	9. Clamper controller
10. Pressure increase controller	11. Pressure increase jar	12. Volume tank
13. Tank bottom valve	14. Flow meter	15. Flow adjusting valve
16 Bottom valve controller		

6.4.12.1 Water meter error testing

a) Clamp the water meters using connectors to install the water meters on the device by the clamper, close flow adjusting valves and open the inlet valve, it should be no leakage in each connector.

b) Open the flow adjusting valve slowly, to exhaust the pipes in normal flow rate of water meter. After the pointer of meter running smooth a few time, close the flow adjusting valve and record the starting reader of water meter Vs.

c) Empty the volume tank, and close the bottom valve after 2 min. open the flow adjusting valve to regulate flow rate.

d) Close the flow adjusting valve when the water level to the pre-concerted of the volume tank, record the reader of the tank Vr after the level rest.

e) Record the end reading of water Ve, using flow formula compute the error:

$$\boldsymbol{\delta} = \left\{ \frac{(\boldsymbol{V_e} - \boldsymbol{V_s}) - \boldsymbol{V_r}}{\boldsymbol{V_r}} \right\} \times \mathbf{100}$$

6.4.12.2 Water Meter Pressure Proof Testing

a) Clamp the water meters using connectors to install the water meters on the device by clamper, open the inlet valve, front and back high pressure valves, open the flow adjusting valve slowly to exhaust the pipe.

b) Close the front and back high pressure valves, switch the pressure increase controller to “increase point“.

c) Open the pressure increase valve slowly and immediately close it when pressure reach the regulate value.

d) Switch the pressure increase controller to “normal “point after testing the pressure will resume by opening the increase valve.

As per IS:779:1994 a meter shall be able to withstand constantly without defects in its functioning, leakage, seepage through the walls or permanent deformation, the continuous water pressure of (i) 1.5 MPa for 15 minute, and (ii) 2 MPa for 1 minute , when tested in accordance with IS 6784:1984.

6.4.12.3 Nominal Capacity Rating

Allow the water to pass through the meter in such a way that the flow rate corresponds to the minimum discharge (for 15 mm dia. Semi-positive 2000 l/hr and for inferential type: 2500 l/hr) and check whether the head loss is within the requirement.

6.4.12.4 Continuous Running Capacity Rating

The same procedure as indicated for the nominal capacity rating shall be followed but the discharge should be 1000 l/hr for semi-positive and 1500 l/hr for inferential type meter dia. 15 mm

For meters with nominal. Flow rate Q3≤ 16 m3/hr, run the meter at its nom. flow rate for a period of 100 hr. and check its accuracy.

6.4.12.5 Minimum Starting Flow (Durability Test)

Allow the water to pass through the meter in such a way that it complies with the minimum starting flow rating i.e. 10 l/hr for semi-positive and 40 l/hr for inferential type 15 mm dia. meter and check whether it starts registering at this flow rate.

6.4.12.6 Temperature Suitability Test

The water meter shall be immersed in a water bath maintained at 45 ± 10 C for 10 hours and then checked for nominal characteristics.

6.4.12.7 Life Test (Wear Capacity Test)

Two unopened meters shall be subjected to the life test by passing through them water at the continuous running capacity rating for a period of 1500 hours, the running period being not less than 8 continuous hours in a day.

After the meters have undergone the life test, they shall again be subjected to nominal capacity rating test and metering accuracy test.

One of the meters which have undergone the life test (preferably the one that has shown greater deteriorating performance under the flow tests) shall be dismantled completely and examined to ensure that there is no undue wear or distortion.

6.4.13 Test Report

A report of test shall be furnished as described below:

Performance Test Report for Water Meter

Meter Makers/ Suppliers		**Meter No.**	
		Dial:	
			Remarks
1. Nominal Capacity rating of Meter: Minimum discharge with head loss not exceeding 10 m			
2. Error at nominal capacity:			
3. Continuous running capacity of meter Minimum discharge with head loss not exceeding 3 m			
4. Error at continuous running capacity			
5. Lower limit of flow			
6. Error at lower limit of flow			
7. Minimum starting flow			
Defects Noticeable After Test (If Any)		Meter Performance (as 1 to 7 above After Test	
8. Temperature suitability test			
9. Hydrostatic test			
10. Life test			

7. SCADA COMPONENTS

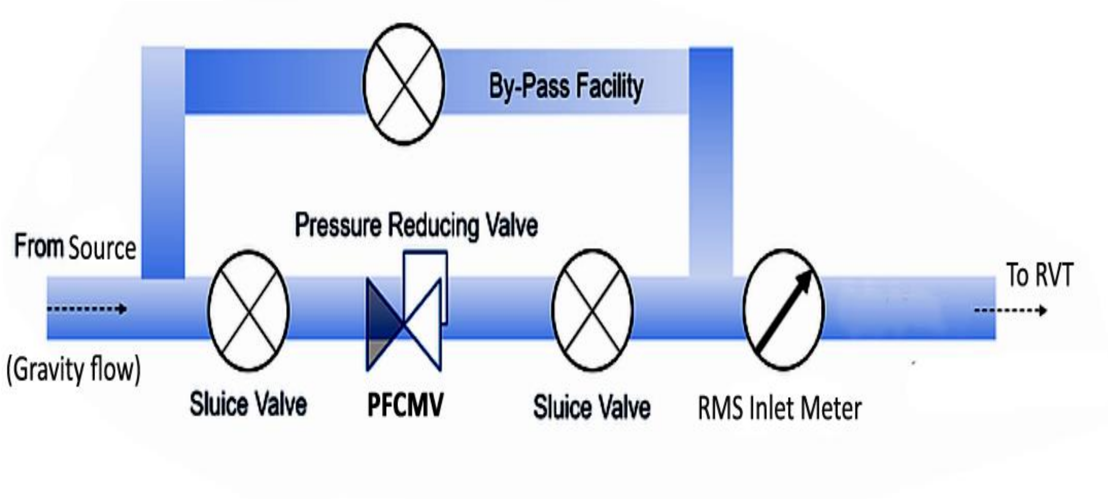
SCADA components are a combination of Smart Water Management System, hardware and software, as SCADA is used in different sectors due to which its components vary according to each sector, for water distribution sector, the component is named according to the location and function performed.

7.1. Components of Smart Water Management Components:

7.1.1. Pressure & Flow Control and Monitoring Device (PFCMV): It shall automatically performs one, two or more independent functions as per the requirements, such as it can be control with the help of an actuator, It can be partially or fully opened or closed with the help of actuators, Anti Draining of System, reducing higher upstream pressure to a constant maximal downstream pressure or sustaining maximum set Flow. All functions are performed irrespective of change in upstream pressure and/or demand. Functions can easily be added or removed in a modular way.

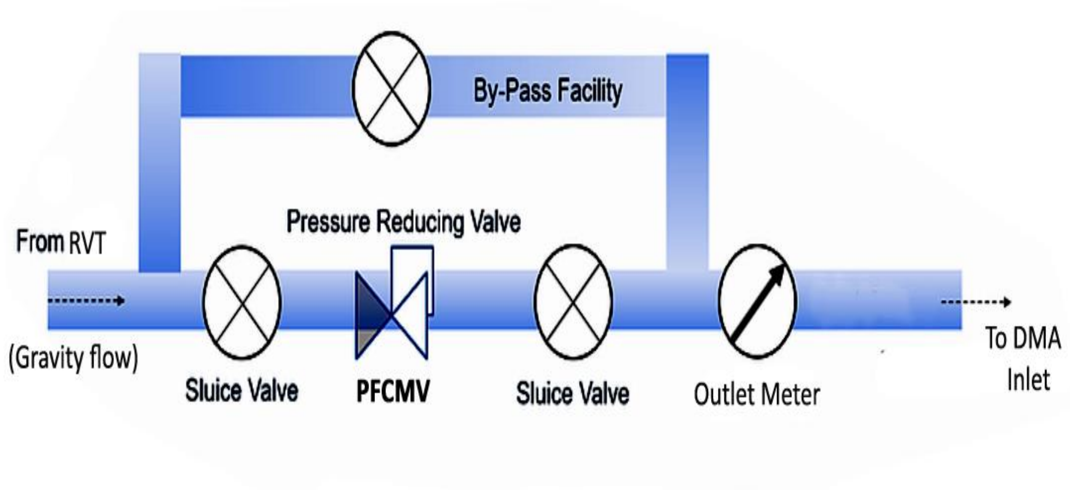
7.1.1.1 Reservoir Management System (RMS): RMS shall be composed of Control Valve, Communication Devices, Pressure and flow metering devices. It is the sub-system of the smart water management. It shall performs one, two or more independent functions as per the requirements, such as

- Prevent any overflow from the reservoirs.
- Reduce water hammer and minimize dynamic pressure effects.
- Provide status and quantity of water available in reservoirs.
- Provide automation between Reservoirs and Inlet source (i.e. Pumps and Gravity flow)



Reservoir Management System (RMS)

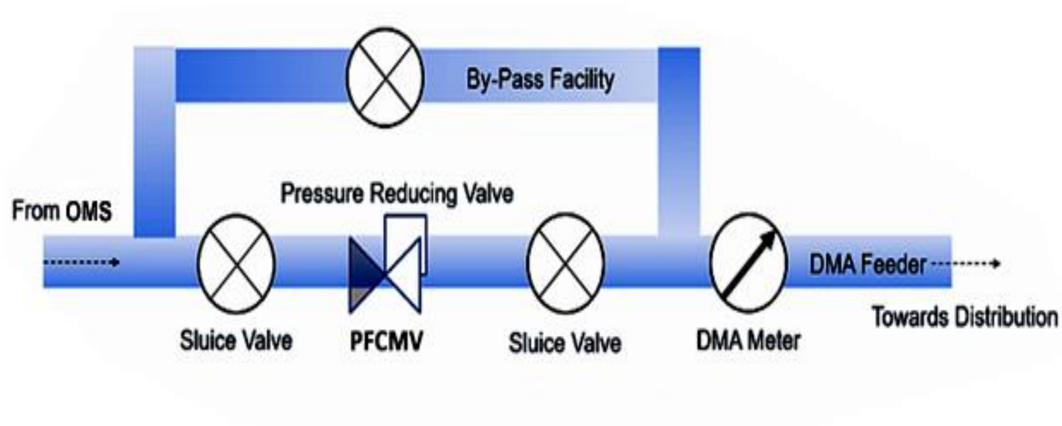
7.1.1.2. Outlet Management System (OMS): OMSis an arrangement of PFCMV, Communication Devices, Pressure and flow metering devices .It is the type of SCADA Hardware part, which sets the water distribution network to adequate pressure setting during high demand hours and reduces the pressure within the Zone/ DMA during low demand hours. During the low demand hours (night hours) the pressure within the Zone/ DMA shoots up, causing increased leakage through the existing breaks and ruptures on pipeline. As the PFCMV reduces the pressure, the resulting leakage is also reduced. This may apply to the inlet point of each metering zone and to the outlet point of the Distribution supply & controlling reservoir according to the scope of the projects. This system is also known as Outlet Management System (OMS).



Outlet Management System (OMS)

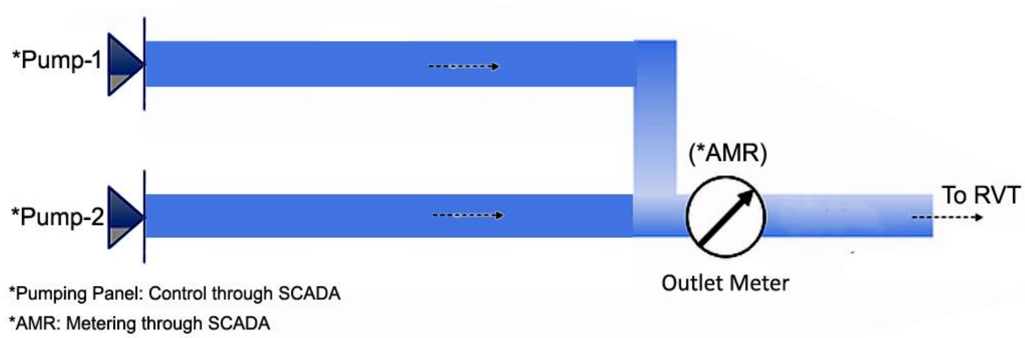
7.1.1.3. Pressure Managed Areas (PMA): PMAis a component of the SCADA system that has controlled pressures. It is the sub-system of Smart water managements that is an arrangement of Pressure flow control metering Valve (PFCMV), Actuator and Communication Devices. Typically, this Sub-system is installed on the Feeder to the DMA at its entry point. This Sub-system arrangement ensures equal pressure within each DMA of the operating area; it is applied to the inlet point of each DMA.

This Sub-system can be set manually to a static pressure setting or can be installed with a Pilot which dynamically operates and sets the Pressure based on the downstream diurnal/ weekly or monthly demand fluctuations of the consumers within every DMA. This system is also known as Sub-Distribution Management System (SDMS) or Village transfer Chamber (VTC).



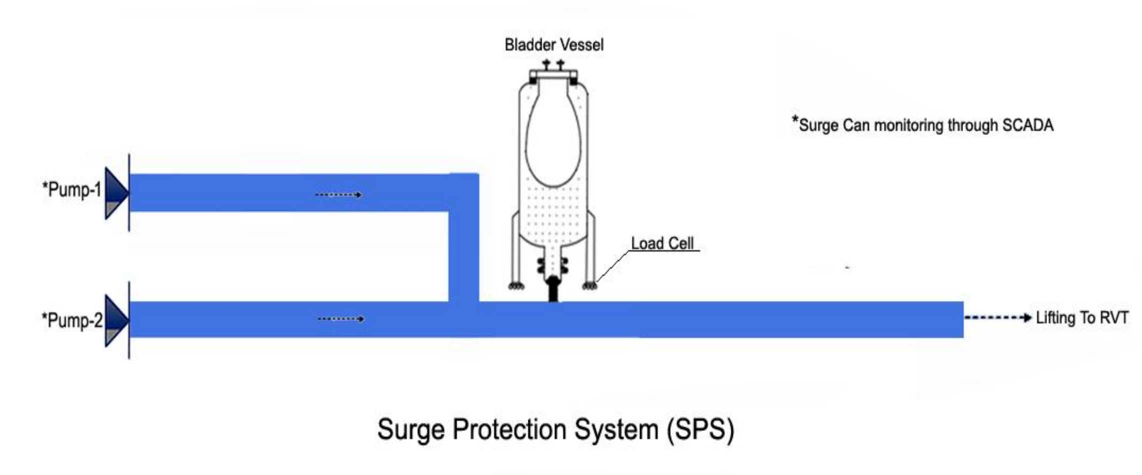
Typical PMA Arrangement at the entry of DMA

7.1.2. Source Management System (SMS): SMS involves Managing, Metering, and controlling the optimum use of water resources. The current method attempts to automate the water management process in the Bore-well, Sump-well, Tube-Well or Stream/River/spring using SCADA logical programming language. The purpose of this study is to reduce water wastage and human effort by partial/complete automation of existing water management system. Systems are connected, monitored and controlled using SCADA. The given solution controls the water resources automatically leading to an efficient water management system. The system executes this task independently while reducing human labour without increase in heavy electricity usage.

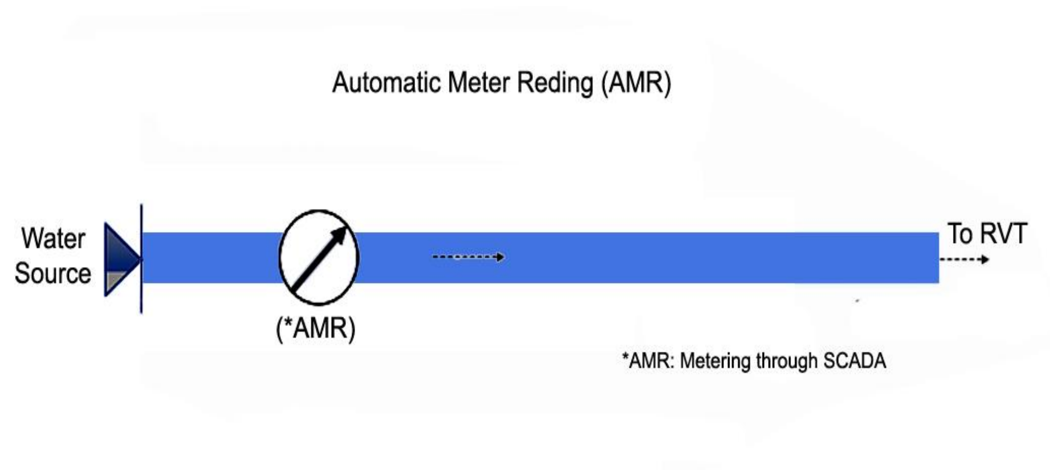


Source Management System (SMS)

7.1.3. Surge Protection System (SPS): The bladder vessels with pressure gauge have been proposed as a measure to mitigate the issue. The steel body encased bladder made of composite neoprene material which shall be filled with compressed air is proposed to be provided at the surge observed points (calculated surge based on the residual head of the pumping main). After the bladder is filled with the desired amount of air it serves as a cushion for the returning water which goes inside the enclosed steel tank and compresses the bladder, which in turn absorbs the kinetic energy of the returning water thus neutralizing the surge effect.



7.1.4. Automatic meter reading (AMR): AMR is the technology of automatically collecting consumption, diagnostic, and status data from water meter and transferring that data to a central database for NRW calculation, troubleshooting, and analyzing. This technology mainly saves utility providers the expense of periodic trips to each physical location to read a meter. Another advantage is that water management can be based on near real-time consumption rather than on estimates based on past or predicted consumption. This timely information coupled with analysis can help both utility providers and customers better control the use and production of water consumption.

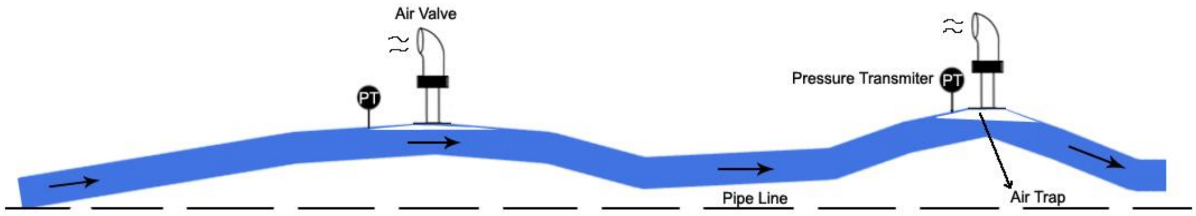


7.1.5. Air & Leakage Management System:

The leak detection system is a very important function for the area of water distribution for which a pressure transmitter is used in the SCADA system.

Air valves shall be used in those parts of DMA distribution where there may be an air trap in the pipe due to elevation differential and due to which the flow of water and its quantity may decline. It also avoids the problem of pipe burst.

Pressure transmitter shall be installed at various locations of the pipeline / DMA to detect leakage based on pressure differential.



*PT: Pressure can measuring through SCADA

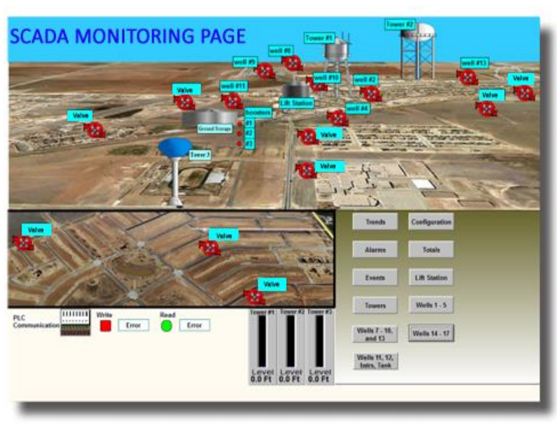
Air and Leakage Management System

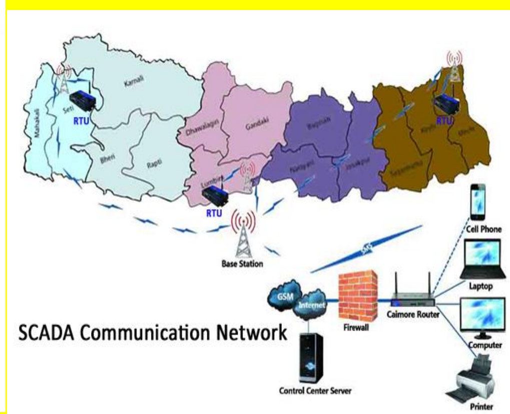
7.2. SCADA hardware components:

SCADA hardware components are combinations of electro mechanical items that includes in the Electrical part: RTU, PLC, sensors, Relay, Actuators, Communication devices, Power supply (SMPS, solar battery etc.) and in Mechanical items: Air Valve, Control Valve, Solenoid Valve, pilot, Flow and Pressure Meter.

7.3. SCADA Software and Monitoring System:

Supervisory Control and Data Acquisition (SCADA) systems are an integral part of Water distribution Management. RTU devices and SCADA software connectivity solutions enable cost-effective monitoring and remote management of critical equipment, in some of the most remote areas.





RTU enables the collection of equipment data from remote SCADA sites and converts it into actionable intelligence using cost-effective and reliable satellite and cellular communications. This makes it easier to reduce maintenance costs and site visits, minimize downtime and maximize output of a variety of critical equipment.

Advantages of SCADA Monitoring System

Gain Full Visibility of Remote SCADA System Sites: Monitor and control critical SCADA system equipment with full visibility from a Master control center via Server based SCADA Monitor Software.

Connect remote, orphan sites, where connectivity is otherwise unavailable, unreliable or cost-prohibitive.

Enhance production by enabling more efficient and cost-effective collection of critical operations data.

Reduce Maintenance Costs and Equipment Downtime: Lower operations costs by enabling preventative maintenance with regular equipment monitoring.

Reduce downtime and production losses with early fault detection and quick response time.

Increase Efficiency and Reduce Site Visits: Reduce labor, vehicle and fuel costs with process automation requiring fewer routine and emergency visits to remote sites.

Deliver alarms and event notifications based on user-defined thresholds and criteria.

Connect to Remote PLCs and RTUs: Provide connectivity between remote SCADA PLCs and the Human Machine Interface (HMI) system using edge analytics to send only relevant information.

Streamline operations by using edge analytics to collect and analyze field data locally.

Bridge the communications gap between RTUs and Data server IP-based SCADA control systems.

7.4. Guidelines for SCADA Detailed Engineering Design:

7.4.1. Field study:

Field study is very important because it helps to gather the information required for the SCADA report (i.e. Coordinates, Photograph and availability of GSM signal), so that the predetermined objectives from the field report are examined according to the report and the problems are studied in depth.

7.4.2. Desk work:

- Data analyzing Reports and secondary data's.
- Field study Report & Photos.
- Signal status on proposed location for GSM based RTU.
- Location of Proposed SCADA components.
- Justification for Proposed SCADA components: Through hydraulic analyzing, it shall be verified that the proposed DMA with SCADA system follow the network planning without interrupting the demand flow of the main network.
- Market study and Cost Estimate

7.4.3. Design:

Source Management:

- If the Stream/River/spring source flows under gravity system to collection chamber / WTP then shall use flow transmitter (AMR) for source metering.
- If the source is bore-well / sump-well / pump / motors, source control shall be done through RTU at relay control panel and for source metering, flow transmitter (AMR) shall be use at the outlet of the pumping line.

Reservoir Management:

If the inlet flow of the reservoir works under the pumping system, then for metering shall be used flow transmitter.

If the inlet flow of the reservoir works under the gravity system, then for metering & Controlling shall be used flow control and metering valve.

Reservoir Outlet Management:

Outlet Control & Metering valve shall use for Outlet line of the Reservoir

DMA Management:

DMA Control & Metering valve shall use for Inlet lines of the DMAs (Up to Bulk Meter)

Air & Leakage Management:

Air valves with pressure transmitter shall be used in those parts of DMA distribution where there may be an air trap in the pipe due to elevation differential and due to which the flow of water and its quantity may decline.

Surge Management:

Bladder vessels shall be used in pumping lines to avoid the water hammer effect on NRV / Pumps, due to transmitting water at high altitudes.

7.5. Things to note for SCADA Design:

- Some Important Points to Implementing SCADA systems into DMA based water distribution System:
- The cost of the SCADA system depends on the DMA design, so the DMA design shall be made a SCADA cost effective system.
- There are many types of SCADA valves available on the market. SCADA valves shall be selected based on the scope and requirement of the DMA network.
- The PRV / SCADA valve shall be used during the main network design so that PRV also can be used in the hydraulic modeling of the network.
- The appropriate medium for communication of RTU shall be ascertained in all proposed locations.

- Elevation Profile for effective air management, weighting the nodal distance for calculating (min, max) pressure values.
- Pressures transmitters shall be used in different locations of the network as per the requirement so that leakage, Water theft can be identify & fix it, on the basis of pressure difference.
- Each DMAs Inlet shall be separated from other one by one or two inlet lines.
- Each DMAs Inlet shall have separate Bulk water meter and SCADA Isolation valve.
- SCADA isolation valve must be capable of pressure regulation, flow control and flow metering (i.e. PFCMV)
- The location of each PFCMV shall be on the side of the road (Social Disturbance Free Zone).
- SCADA Component is a combination of Hydro-electro mechanical items, electronic items have to be secured by a waterproof cabinet or chamber.

8. SANITATION

Sanitary latrines and low cost drainage for draining surface run-off can be included in the Project design, where no treatment is necessary or required.

8.1 Household Drainage

Household drains can either connect to main street drains or simply channelize the surface run-off to a potential outfall like a natural drainage or pond.

8.1.1 Hydrological Analyses

8.1.1.1 Estimation of Runoff

Rainfall is one of the prominent variables for runoff generation. During occurrence of rainfall, various factors prevent rainfall to reach over the ground like vegetation, building and other objects. Similarly, some amount of rainfall also lose by infiltrates into the soil, and rest of the water amount makes a head over the surface, which moves from one place to another under the gradient effect, and ultimately meets to the streams, channels etc. called as runoff.

As a result of urbanization, imperviousness in watershed area is increasing substantially. Impervious surface include road, sidewalks, parking, building etc. As a result, natural flow paths in watershed is replacing or supplementing by paved gutters, storm sewers, and other elements of manmade drainage system. Which cause many problems like increases erosion, discharge and volume, which emerge as a most challenging part for the hydrologist to design and implement measures that will minimize its adverse effects. Hydrological study to determine runoff and peak discharge is based on long-term stationary discharge records of the watershed. Such precise, information's are rarely or scantly available. Even though, where they are available, for doing accurate statistical analysis on it is usually challenging job because of conversion of land to urban uses during the period of record. Therefore, catchment characteristics become prominent variables for the estimation of peak discharge and through understanding of the characteristics and experiences the importance of variables can make sound judgments on how to alter variables to reflect changing watershed characteristics.

Urbanization significantly changes watershed response to precipitation. The most significant effects are reduced infiltration and decreased travel time, which substantially increase peak runoff. Runoff is estimated primarily by the amount of precipitation and by infiltration characteristics related to soil type, soil moisture, antecedent rainfall, land cover type and surface retention. Travel time is determined primarily by slope, length of flow path, depth of flow, and roughness of flow surfaces. Peak discharge is based on relationship between these variables and watershed area.

The Rational Method is the most commonly used method for the determination of the runoff. The following formula is used for the estimation of the run off by Rational Method.

$$\mathbf{Q = C\ i\ A}$$

Where,

Q = discharge in m^3/s

C = runoff coefficient

i = rainfall intensity in m/s

A = catchment area in m^2

Rational Method is fairly tested and holds true for urban areas in Nepal. This method assumes that the maximum runoff for a particular catchment will occur when runoff from all of the catchment is contributing. The determination of the runoff by the Rational Method requires the values of runoff coefficient, rainfall intensity and contributing catchment area. A rainfall intensity for the duration corresponding to the time of concentration which represents the maximum runoff should be estimated.

8.1.1.2 Runoff Coefficient

The runoff coefficient represents the fraction of the total rainfall that is available in the form of storm water flow or runoff reaching a sewer. Its value depends on the imperviousness and the shape of the catchment area or drainage area, and the duration of the storm. The runoff coefficient increases with the increase in the imperviousness of the catchment area or drainage area, because greater is the imperviousness of the area lesser will be infiltration and hence greater will be runoff.

One of the major factors in the use of the Rational Method is the judicious use of the coefficient of runoff "C". Potential future growth of the area within a town needs to be considered while selecting the runoff coefficient as the coefficient is likely to increase with growth. Therefore, a lot of engineering judgment is involved in determining "C". The value of runoff coefficients for various types of land use types are shown in Table 25.

Rational formula assume homogenous watershed, but in actual practice it may not happen and different surfaces have its own runoff coefficient values. Therefore, weighted equivalent runoff coefficient Ce is computed from the following formula to represent average watershed runoff coefficient.

$$C_e = \frac{\sum_1^n C_i A_i}{\sum A}$$

Table 25: Runoff Coefficients

Land Use	Runoff Coefficient	Land Use	Runoff Coefficient
		Lawns:	
		Sandy soil, flat, 2%	0.05 - 0.10
Business:		Sandy soil, avg., 2-7%	0.10 - 0.15
Downtown area	0.70 - 0.95	Sandy soil, steep, 7%	0.15 - 0.20
Neighborhood area	0.50 - 0.70	Heavy soil, flat, 2%	0.13 - 0.17
		Heavy soil, avg., 2-7%	0.18 - 0.22
		Heavy soil, steep, 7%	0.25 - 0.35
		Agricultural land:	
		Bare packed soil	
		Smooth	0.30 - 0.60
		Rough	0.20 - 0.50
Residential (urban):		*Cultivated rows*	
Single family area	0.30 - 0.50		0.30 - 0.60
Multi-units, detached	0.40 - 0.60	Heavy soil, no crop	0.20 - 0.50
Multi-units, attached	0.60 - 0.75	Heavy soil, with crop	0.20 - 0.40
Residential (suburban)	0.25 - 0.40	Sandy soil, no crop	0.10 - 0.25
Apartment areas	0.50 - 0.70	Sandy soil, with crop	
		Pasture	0.15 - 0.45
		Heavy soil	0.05 - 0.25
		Sandy soil	0.05 - 0.25
		Streets:	
Industrial		Asphaltic	0.70 - 0.95
Light areas	0.50 - 0.80	Concrete	0.80 - 0.95
Heavy areas	0.60 - 0.90	Brick	0.70 - 0.85
Parks, cemeteries	0.10 - 0.25	Unimproved areas	0.10 - 0.30
Playgrounds	0.20 - 0.35	Drives and walks	0.75 - 0.85
Railroad yards	0.20 - 0.40	Roofs	0.75 - 0.95

8.1.1.3 Rainfall Intensity

The first consideration in the design of drainage systems is the estimation of the rainfall intensity. If adequate records of previous historic rainfall are not available, the estimates are made using various methods but nearly all rely on obtaining the design rainfall intensities. Therefore the rainfall intensity duration frequency (IDF) curves will have to be developed for the particular area under design, if needed.

Annual series rainfall frequency curves are most commonly used. Rainfall frequency curves using the 24 hour recorded rainfalls at various rainfall stations will be derived.

For urban design work shorter duration rainfalls are used so that pluviograph records can be obtained to supplement the one-day records, which enable to interpolate the results for shorter duration rainfall events.

The rainfall intensity duration frequency (IDF) curves are developed with the daily rainfall data using Gumbel probability distribution. The typical IDF curves are shown in the Figure 63.

8.1.1.4 Effective Rainfall

During a storm not all of the rainfall enters into the drainage system. That part of the rainfall, which contributes to runoff into the drainage system, is termed as effective rainfall. The rest is termed as losses due to evaporation, evapo-transpiration, infiltration and surface storage etc.

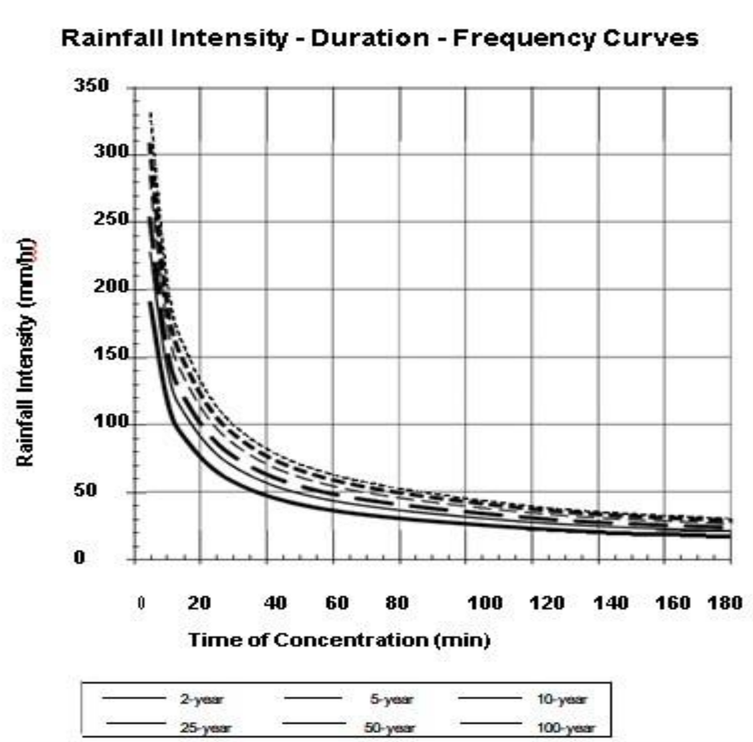


Figure 64: Intensity Duration Frequency Curves

Estimates of rainfall losses need to be made to derive effective rainfall but the most reliable method is to use recorded rainfall and runoff i.e. measuring volume of rainfall and runoff with the difference being attributable to losses. Loss rates are dependent on factors like ground permeability, topographic slope, surface vegetation, rainfall intensity and duration etc. The most useful design tools for design flood estimation are historic recorded data on rainfall and floods.

Flood records can be available from the Department of Hydrology and Meteorology. But most of the records and stream gauging stations are in the upper river reaches of the major rivers. For urban drainage one should obtain flood records (i.e. levels of the most serious floods) in the particular town and adjacent outfall streams from direct field surveys, local inquiries etc.

8.1.1.5 Time of Concentration

Time of concentration is the time at which all of the catchment contributes to a particular discharge point. Time of concentration can be calculated using following equation.

$$\boldsymbol{T_c = L/(V \times 60) + t_{ce}}$$

Where,

Tc = total time of concentration (min)

L = the maximum length of travel for the runoff (m)

V = assumed velocity of overland flow (m/sec.)

tce= assumed extra time of concentration (min)

Although graphical solutions are available to determine the overland flow velocity "V", it is recommended in literature that a value between 0.60 m/s to 1.5 m/s can be used. Overland flow velocity can be adopted based on the general topography of the area under consideration. Similarly, "tce" is usually assumed at 0.25 hrs (15 min.) as per convention.

Similarly, the well-known relation developed by USDI as shown below can also be used for the determination of time of concentration

$$\boldsymbol{T_c = 57\left(L^3/H\right)^{0.835}}$$

Where,

Tc = Time of Concentration (min)

L = Maximum Length of travel of water (in Kilometer)

H = Difference in elevation of catchment (meter)

8.1.1.6 Economic Selection of the Design Flood

The optimum design flood for an urban drainage system is chosen in most instances on economic considerations. The choice is made on the basis of providing a cost-effective system which will discharge the storm and flood waters adequately for most rainstorm situations.

Quite often it is impractical to alleviate all flooding conditions since a large capital investment required and to alleviate extremely large floods may not be economically justified. It is often the practice to provide for complete relief from smaller floods and at the same time have partial protection from larger flooding situations.

Therefore, the design flood will be derived by considering the recurrence interval of floods (and rainfall) and designating the design flood with that recurrence interval. Normally, the recurrence interval is selected to provide the optimum benefit cost ratio for a series of recurrence intervals.

Rainfall patterns in Nepal provide generally good rainfalls (mostly in the monsoon season) with typical annual rainfalls of 1200 to 2000 mm per year. However, the following return periods are recommended to ascertain the design rainfall.

Once in two years	-	for storm water drain design
Once in five years	-	for culvert design

In all cases checks should be made to assess the impacts of a larger flood; the criteria being that overtopping should not cause major or disastrous situations. Thus, for culverts a check should be made to see the effects of 1 in 10 years flood.

8.1.1.7 Check for Adverse Effects

Finally, the drainage system design will be checked for adverse effects like backwater, storage problem, partial flow in pipe culverts etc. The following paragraphs briefly elaborate the above mentioned effects.

Backwater Effects

The problem in storm water and sewerage design occurs in flat terrain where the design water level is controlled not only by the size and slope of the drain but also by downstream backwater. This may be due to a river or lake into which the drainage system enters.

To design for backwater effects we shall estimate or obtain historic flood levels, which will affect storm water runoff in the drain. With these established control water levels we calculate the water profile in the drain using backwater computational methods. In this regard, the use of hydraulic software like the HEC-2 and /or HEC-RAS is recommended to developed backwater water profiles for various scenarios.

Storage Effects

Another problem, which is due to storage effect, shall also be considered. If the required drain is blocked or enters a large storage area such as a lake or large pond, the lake or pond may have a retarding effect on the flows, similar to a dam of a reservoir.

To estimate whether there are storage effects, the reservoir routing method may be carried out to check for any retardance of design flows by a lake or pond.

An inflow hydrograph will be developed for the inflow and storage data developed for the active part of the lake or pond storage, if relevant information is available.

Partial Flow in Pipe Culverts

For pipe culvert designs, a further addition to the calculations will be made to check the actual versus the design flow in a pipe culvert. This is to check actual flows rather than design flows. Quite often the finally adopted longitudinal slope or drain size may be the one chosen in the design.

The main check will be on velocity and total head loss and to ensure that the velocities and water levels are within the design criteria.

8.1.2 Hydraulic Assessment and Design

The drainage system will consist of either pipes and box drains, road gullies and rain inlets, manholes or culverts etc. The hydraulic design and / or assessment in terms of their conveyance capacities of these structures are briefly elaborated below.

8.1.2.1 Pipe and Box Drains

The drain size will be established according to least cost and velocity criteria using

Manning's formula:

$$\boldsymbol{Q = \frac{1}{n} A R^{2/3} S^{1/2}}$$

Where,

Q	= Design Discharge (Cu. M /s)
A	= Water way area (Sq. m)
R	= Hydraulic Radius (Area / Wetted perimeter) (m)
S	= Longitudinal Slope (dimensionless)
n	= Mannings Coefficient of friction

The following Manning's 'n' values are recommended for various materials:

Pre-cast concrete pipes	:	0.013
Cast in-situ concrete drains	:	0.015
Concrete lined drains	:	0.015
Brick or stone masonry cement render	:	0.015
Brick or stone masonry unrendered	:	0.018
Rocks or broken bricks etc.	:	0.02
Unlined excavated channel	:	:0.031

The recommended limiting design velocities are as:

Lined pipes and drains

Minimum	=	0.6m/s
Maximum	=	3.0 m/s
Unlined drains		
Minimum	=	0.6 m/s
Maximum	=	1.0 to 1.5 m/s, depending on soil conditions.

The minimum flow velocity will, however, be based on providing flushing or self-cleansing flows and avoiding siltation. For round pipe drain design flow capacity will be checked for different flow conditions i.e., whether full flow or partial flow.

The depth of water in the drain at maximum flow should not be more than 90 percent of the depth of the drain.

Pipe drains on road are used where space is available and closed drain needed; a minimum cover is generally required. The minimum and maximum pipe sizes to be use shall be 300 and1400 mm RCC pipe respectively. In principle due to the diameter, all pipes to be used in the drainage/sewer network are proposed to be RCC pipes with spigot and

[1] Note: More detailed values for unlined drains and natural streams will be referred to text such as VenTe Chow's "Open Channel Hydraulics"

socket ends, jointed through rubber gaskets (NP3). The minimum cover above the crests of the pipe is to be 1.0 m but deep sewers are the inevitable result of relatively flat catchment/gradient.

8.1.2.2 Road Gullies and Rain Inlets

Flows on the roadway should be directed to a drain with the correct road profile design both longitudinal and cross fall. For urban roads with roadside kerbs and drains the following road grades are recommended to ensure that water will flow off the road into an inlet:

Cross Fall (asphalt)	=	3 %
Longitudinal road slope	=	preferable minimum 0.5 %
	=	absolute minimum 0.25 %

The most efficient gullies to discharge water from the road into a drain are the horizontal (grill type) gullies. Vertical inlets, which require some head are less useful. A dead depth (depth of bottom level of rain inlet chamber from pipe invert) of minimum 20 cm shall be kept to allow silt to settle. The spacing of the gullies or rain inlets shall be according to their capacity design. However, they shall be placed at the lower point of the road surface on both sides of the road.

8.1.2.3 Manholes

Manholes must be provided for underground drains at appropriate intervals to ensure easy access for cleaning of the drains.

The manhole is an obstruction to flow in a drain line. Therefore head loss allowances should be provided at each manhole. A head loss equal to k $v^2/2g$ should be added to the friction loss at each manhole. Typical values of the coefficient "k" for two way and three way junctions may be used as given below:

For straight through manholes	k	=	0.5
For 45 to 90 degree junctions	k	=	0.8
For junctions greater than 90 degrees	k	=	1.5

For three way junctions; the major pipeline and flow condition as for two way junction with an additional allowance of 25% loss may be used. In case of manhole where branch sewer meets the main sewer line, the outlet pipe should depressed by at least 2 cm from the inlet pipe.

8.1.2.4 Culverts

Culverts will be assessed and / or designed according to whether flow is inlet controlled or outlet controlled. In case of inlet controlled the flow through the culvert is controlled by friction and geometric arrangement at the upstream end of the culvert (wing walls etc.). The discharge capacity is expressed in terms of:

H/d

Where, H = Total head at inlet above the invert level.

d = Diameter or size of opening.

Whereas the flow in an outlet controlled culvert is controlled by the downstream water level or by the friction along the culvert walls.

The capacity of a culvert with outlet controlled (sometimes referred to as outlet) is obtained by estimating the head losses at the inlet, outlet and along the barrel of the culvert.

Inlet and outlet losses are expressed as a function of:

$$\frac{k(V_{1^2}) - V_{2^2}}{2g}$$

Where,

V1 = Velocity into the inlet (or outlet)

V2 = Velocity out of inlet (or outlet)

k = inlet or outlet loss

The values of entry and exit losses for different entry and exit conditions as shown in Table 26 may be used.

Table 26: Entry and Exit Losses

Entry/outlet Condition	K_{in}	k_{out}
Rounded transition (e.g. stone masonry)	0.2	0.3
30^{o} wingwalls to box culvert	0.3	0.5
45^{o} wingwalls to box culvert	0.5	0.8
Square entrance to box culvert	0.75	1.0

Barrel losses are estimated applying the Manning's equation; the friction loss along the barrel is = S x L. (where S is the longitudinal slope of the water line and L is the barrel length).

The sum of the entry exit and barrel losses is the total head loss across the culvert. By a series of trial and error calculations one can establish the required culvert size or assess the capacity of existing culverts according to the amount of head loss that is available or acceptable in each particular situation.

8.1.2.5 Lined and Unlined Drains

The drains can either be lined or unlined and the main drains are often constructed as open drains. This is usually a matter of economics and the selection whether to provide lining or not is dependent on:

- Hydraulic considerations
- Cost
- Public acceptance and ease of maintenance etc.
- Location

Lining of drains provides a smoother flow. Normally, a lined drain has approximately double the flow capacity of an unlined drain and therefore is a strong consideration in flat areas where we may need to restrict the size of a drain.

For all major drainage lines located outside the settlement area, provision shall be made to provide a berm along the edge of the drain which can be used as a maintenance track. The berm width in these cases shall be approximately 1.0 meter. The drainage lines within the market area are however shall be kept closed as far as possible to avoid those becoming solid waste dumping pits. In order to avoid drainage lines becoming undersized due to increase in the discharge as a result of urbanization, runoff coefficients during the design shall be adopted on a higher side (0.6 to 0.8).

8.1.2.6 Identification of Drainage Bottlenecks

Besides establishing design parameters and criterion and assessing the hydraulic capacities of existing and proposed infrastructures, it is imperative that bottlenecks in efficient drainage flow are to be identified. In this regard the entire drainage pattern and characteristics of the area under consideration including potential areas for urban expansion shall be thoroughly reviewed. A field reconnaissance needs to be carried out by the drainage engineer in conjunction with the hydrologist for the entire catchment area. The base map with the necessary topographical details and catchment demarcations shall be used during the field reconnaissance. On the site verification, historical records and local inquiry shall form the basis for such an identification process.

This information shall be further augmented by collection of data / information on frequency and extent of water logging in various areas. The extent of the problem relating to discharge of pollutant discharges by the local industries on to the natural water-ways and drains needs to be thoroughly assessed. These shall be done primarily on the basis of existing records, technical literature / documents and local inquiry.

Before designing a drainage system for a town partially or wholly, a conceptual master plan must be developed first with proper outlet/s (discharging point/s). Otherwise, drainage lines designed in isolation can be undersized and create problem of flooding in future.

8.2 Sewerage System

8.2.1 Types of Sewerage System

The entire system of conduits and appurtenances for collecting sewage and delivering it to a disposal point is called sewerage system. Sewerage systems are of three types:

1. Separate system
2. Combined system
3. Partially separate system

8.2.1.1 Separate System

In this system two separate sets of sewers are installed, one for carrying sanitary sewage and other for storm water. Open drain can also be used instead of sewer to carry storm water.

8.2.1.2 Combined System

In this system both sanitary sewage and storm water are carried in a single sewer.

8.2.1.3 Partially Separate System

In partially separate system, in addition to sanitary sewage, a portion of storm water is allowed to enter in the sewers carrying sanitary sewage. The remaining portion of storm water is diverted into separate set of sewers or open drains.

Present-day construction of sewers is largely confined to the separate system except in those cities where combined systems were constructed many years ago. In newly developing urban areas the first need is for collection of sanitary sewage, and, since sanitary sewers are relatively small and inexpensive, they can usually be constructed without long delay. The sewerage system shall be of separate system type where sanitary sewage and storm water carried separately.

Before developing a conceptual master plan, which is mandatory to avoid under/over sizing of individual sewer lines, the catchment area of the existing settlements including areas of potential urban expansion is thoroughly assessed and analyzed.

8.2.2 Quantity of Wastewater

The total quantity of wastewater generated in a town is the sum of the domestic wastewater, nondomestic wastewater and groundwater infiltration.

8.2.2.1 Domestic Wastewater

In estimating current and future domestic wastewater generation rates and volumes, the two key data sources are (i) population estimates, and (ii) the per capita wastewater generation. The latter is a function of assumptions made regarding the return rate from water consumption.

The wastewater return factor is in the range of 0.7 and 0.8 ("Manual on Sewerage and Sewage Treatment", Ministry of Urban Development, November 2013). It is expected that water distribution will improve to cater to a larger percentage of the population. In parallel with the improvement in water supply, sewerage coverage will be extended to more households; thereby higher percentage of the population will be able to use flush toilets. Therefore, the return factor shall be taken as 0.8, to reflect higher discharge of wastewater from improved sanitation facilities and water supply conditions. Considering the return factor of 80%, the quantity of sewage will be 80% of the water consumed in the town.

8.2.2.2 Peak Factor

For the purposes of hydraulic design of sewer stretch, the estimated peak flow is to be adopted. The peak factor or the ratio of maximum to average flows depends upon contributory population and is then applied to the average daily flow. The sewage flow in sewer varies considerably from hour to hour and also seasonally. Dry weather flow varies during the day with a major peak typically occurring in the early morning.

This peak is dependent upon the number of inhabitants as well as on the size of the catchment area. As the catchment areas expands, the peak value decreases due to the superposition of different dry weather flow hydrographs and flow attenuation in the network. The methodology followed by the Harman formula takes this into account. The peak factor to be adopted for various contributory population in a sewer stretch is shown in the Table 27.

Table 27: Peak Factors for Sewer Design

S.No.	Contributory Population of Sewer Stretch	Peak Factor
1	Up to 20,000	3.00
2	20,000 – 50,000	2.50
3	50.000 – 750,000	2.25
4	More than 750,000	2.00

8.2.2.3 Nondomestic Wastewater

The nondomestic wastewater includes the wastewater generated from various industries and commercial establishments. The nondomestic wastewater is proposed to be taken as 10% of domestic wastewater. This value has been already adopted in the design of sewers in Urban Environment Improvement Project (UEIP) in Nepal.

8.2.2.4 Groundwater Infiltration

It is appropriate and conventional to allow for a certain amount of ground water infiltration into the sewerage system which must be reflected in the hydraulic design. The quantity will depend upon workmanship in laying the sewers, level of ground water table, material of the sewer, soil type, condition and nature of sewer joints, etc. The Manual on sewerage and Sewage Treatment, Ministry of Urban Development, India (2013) has stated that the design infiltration value shall be limited to a maximum of 10% of the design value of the sewage flow.

With improved standards of workmanship, quality and availability of various construction aids the infiltration should tend to the minimum rather than the maximum. The use of HDPE pipe with less jointing of better quality than RCC pipes could further reduce the infiltration factor. Based on prudent engineering practice, 2 to 5% of dry weather flow shall be considered as for infiltration into the sewer network.

8.2.3 Velocity of Flow

Sewage contains large amount of organic and inorganic solid matters, which remain floating or suspended due to the flow of sewage in the sewer. If the velocity of flow is low, the floating and suspended solids will get deposited on the bottom of the sewer and will go on accumulating, which may reduce the pipe sectional area and will cause obstruction in the flow of sewage. Therefore, while designing the sewer the velocity should be kept such that no solid gets deposited in the sewer. Higher velocity also may scour the pipe surface and decrease the durability.

Minimum velocity of flow in sewer shall be a self-cleansing velocity, which shall be achieved at least once in a day during peak flow at ultimate peak flows. The maximum velocity is restricted to just below the scouring velocity which is 3 m/sec as recommended.

- Minimum velocity: 0.6 m/s at peak flow
- Maximum velocity: 3.0 m/sec

In the Initial stretches of sewer network, it may happen that the velocity be less than self-cleansing velocity due to flat gradient and also less sewage flow. In order to get self-cleansing velocity in the starting stretches of the sewers, the design requires sewers to be laid at greater depth, which is uneconomical. Therefore self-cleansing velocity may not be always achieved at initial stretches and flushing may be required more frequently at this part of the scheme.

From consideration of ventilation in waste water pipes, sewers shall not be designed to run full. The maximum permitted depth adopted in sewers sewer is 0.80% of the pipe high peak flow.

8.2.4 Design Formula

Design practice is to use Manning's Formula for open channel flow and the Hazen Williams and Darcy-Weisbach formulae for closed conduit or pressure flow (*Source: Manual on Sewerage and Sewage Treatment (Third Edition), Ministry of Urban Development, November 2013).*

The sewers are not designed as pressure flow pipes except in the inverted siphon. The sewers does not run full even at the peak flow. The Manning's formula is widely used in the design of sewers. The manning's formula is given by the following equation.

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where;

Q	=	discharge (m^3/s)
S	=	slope of hydraulic gradient
R	=	hydraulic radius (m)
A	=	cross sectional area of flow (m^2)
V	=	velocity (m/s)
n	=	Manning's coefficient of roughness

The value of n shall be adopted as 0.013 for RCC pipe and 0.009 for HDPE pipe.

Sewers will be designed in such a way that the depth of flow in the sewer at peak flow is not more than 0.8 times the diameter of the sewer i.e. $d/D \leq 0.8$. It is not possible, in most of cases, at head of sewer network to achieve the required depth due to nominal inflow of sewage.

8.2.5 Sewer Size

The minimum pipe diameter of sewer is ND 200 mm and ND 100 mm for House connection. For commercial connections such as Hotel the pipe diameter for the connection is ND 150 mm. The connection from the service pit to the sewer manhole is ND 150 mm. The maximum pipe diameter of sewer shall be ND 1600 mm.

8.2.6 Ruling Gradient

A minimum sewer gradient shall be 1 in 200. Since in most of the starting reaches sewage quantity is very less and adequate velocity is not developed, sewers shall be laid at a minimum slope of ruling gradient. Ruling gradient refers to the slope at which the sewage with design depth would develop self-cleansing velocity.

8.2.7 Pipe Materials and Joints

In principle all pipes to be used in the main sewer network are proposed to be reinforced concrete cement RCC pipes with spigot and socket ends, jointed through rubber gaskets. RCC pipes shall be manufactured as per NS80/2042 and IS 458:2003 medium duty, non-pressure pipes. The cement to be used for RCC pipes shall be ordinary Portland cement as per standard: The jointing shall be Rubber Ring Joint (RRJ) type and the rubber gasket to be provided shall be as per NS, NP3 or exceptionally NP4 RCC (Hume) pipes for sewer lines as applicable.

Alternatively, High Density Poly-ethylene (HDPE) pipe shall also be used for lateral of branches sewers lines ND 200 and 250 mm for collector, branch and lateral sewers. HDPE pipes have the advantage of not been attacked by sewerage, provides less jointing of better-quality reducing infiltration, and can have lower minimum gradient with a smoother internal surface. HDPE provides durability, performance, easy handling, and toughness during installation and simplified field fabrication. Jointing shall be made by butt welding employing fusion.

8.2.8 Manholes

A manhole is an opening constructed on the alignment of a sewer at frequent intervals to provide access for maintenance purposes like inspection, testing, cleaning and removal of obstructions from the sewer line etc. Manholes are located at each sewer junction or change of sewer direction, diameter or gradient. No connection pipe should enter the manhole at an angle greater than 90° to the direction of the flow.

Manholes are generally constructed in circular shape. The manholes are in brick masonry walls with plastered inside, RC concrete bottom and top slab with cover and finished with a benching. Alternatively small size masonry manholes can be rectangular for easy construction.

Where the water table is high and the pipe sewer is laid in open field or along the side of the drain, the RCC circular manholes would be constructed to avoid any leakages into or out the manholes.

The final level of manholes would be depending on site condition, and therefore the final levels will be finalized during the execution time. However, generally the following values may be considered;

- Paved areas cover level = final paved level.
- Landscaped areas cover level = final ground level +0.1m.
- Open, unpaved areas cover level = final ground level +0.25m or as instructed by the Engineer during construction.
- Manhole covers with frame with a minimum clear opening of 600 mm

Cast Iron (CI) manhole cover and frame should be used for covering manhole, heavy or medium duty CI covers shall be used in relation to the location of the manhole (road or sidewalk).

It has been proposed to provide the manholes at every junction, change of alignment/grade or 30m whichever is less in order to construct the house connections. In large street larger than 4 meter lateral sewerage pipes lines may be laid and the sewerage collection made from service pits, in this case the distance with manhole may be increase from 30 to 60m case by case.

8.3 Wastewater Treatment

The wastewater must be treated to reduce the impurities so that it can be safely disposed into land or water bodies without causing its pollution and hence thereby reducing adverse impacts on health. The degree to which a wastewater is treated depends on the water quality of the receiving body of water into which the effluent is discharged. The required effluent quality can be achieved by combining a variety of different treatment processes. No single process will suit all circumstances and the engineer must select the combination of systems which will provide the desired treatment at minimum cost and with maximum reliability. Therefore, an appropriate wastewater treatment plant shall be worked out to control some essential wastewater parameters line BOD and SS. For treatment of wastes using natural biological methods like stabilizing ponds or oxidation ponds or reed bed (constructed wetlands) techniques can be explored.

8.3.1 Wastewater Characteristics

The untreated domestic wastewater consists of a number of physical impurities such as suspended and dissolved solids, chemical impurities such as nitrogen, carbon, phosphorous, etc., and organic impurities such as BOD and COD. The pH for all these wastes will be in the range of 6.5 to 8.5 with majority being slightly on the alkaline side of 7.0. Three typical compositions of untreated domestic wastewater are summarized in Table 28.

Table 28: Composition of Untreated Domestic Wastewater

Constituent	Weak	Medium	Strong
	(all in mg/l except settleable solids)		
Alkalinity (as $CaCO_3$)	50	100	200
BOD (as O_2)	100	200	300
Chloride	30	50	100
COD (as O_2)	250	500	1,000
Suspended solids (SS)	100	200	350
Settleable solids, mL/L	5	10	20
Total dissolved solids (TDS)	200	500	1,000
Total Kjeldahl nitrogen (TKN) (as N)	20	40	80
Total organic carbon (TOC) (as C)	75	150	300
Total phosphorous (as P)	5	10	20

The disposal of untreated domestic wastewater into surface water will pollute the water bodies. The proposed sewage treatment plants should produce effluent as per the generic standards, tolerance limits for wastewater discharged into inland surface water from wastewater treatment plant set by the Ministry of Science, Technology and Environment.

8.3.2 Design Criteria of Waste Stabilization Pond Treatment System

Waste stabilization ponds, also known as *oxidation ponds*, are the simplest biological systems available for the treatment of wastewater, more practically when a large area of land is available for such treatment. They are employed for the treatment of both domestic wastewaters and industrial wastewaters, which are liable to biological treatment. The ponds are generally constructed in earthwork with relatively very small depths as compared to their large surface areas and bunds (embankments) are built all around to some height to exclude the entry of rainwater into the ponds. Normally wastewater to be treated is applied directly to the pond(s) after removing floating materials through bar racks without any primary treatment. The oxygen required for aerobic decomposition of organic solids is mostly supplied by the algae present in the system through the symbiotic actions of algae and bacteria. The removal of grit is recommended by grit chamber to sludge removal period from the pond. The system has low construction and negligible operating cost as it requires minimum operation skills, and does not use any mechanical equipment to supply oxygen by aeration. Ponds may be multi-celled and can be provided in series or parallel. Accumulation of sludge at the bottom of pond is negligible, only few centimeters, in a year and therefore, requires its removal once in 4 to 5 years.

When raw wastewater is fed to the basin after screening, the suspended solids settle to the pond bottom by gravity due to long retention time. The soluble organic matter in upper top and intermediate layers are decomposed (oxidized) under aerobic and facultative conditions to carbon dioxides (CO2), nitrates, orthophosphate and water by the microorganisms (predominately bacteria) present in the waste. The required oxygen is supplied by the photosynthetic metabolism of algae present and synthesized in the pond. The solids settled at the bottom of ponds are decomposed to stable end products by anaerobic bacteria. Ammonia from the bottom zone is oxidized to nitrate, sulphur dioxide and hydrogen sulphide gas if *Nitro bactre*, *Nitro so bacteria* and *Thiobacillus* microorganisms are present in the system.

The waste stabilization ponds consist of anaerobic ponds and facultative ponds connected in series. Administrative cum laboratory building, watchman/workers building and parking areas have been provided within the premises of the wastewater treatment site.

Anaerobic Pond

This pond is used for the pre-clarification and is the first clarification stage of this pond system. The solid substances build sediment at the base of the pond. There they are biologically decomposed. This pond is generally 2.5-5.0 m deep in which anaerobic conditions prevail throughout the pond depth except for a surface zone of few centimeters. Usually these types of ponds are installed with facultative ponds in series. Stabilization of organic solids involves precipitation and decomposition of organic matter and removal of soluble BOD5 is generally up to 85%.

Facultative Pond

This pond is generally 1.0-2.5 m deep and three zones exist throughout the pond depth, viz. aerobic zone at the surface, anaerobic zone at the bottom and facultative zone at the mid depth of the pond. Stabilization of organic solids is achieved by aerobic bacteria in the aerobic zone, by anaerobic bacteria at the bottom of pond and by facultative bacteria in the middle zone. During the retention period of the wastewater, the solid substances form sediment on the bottom. The pond is to be placed in an area where sufficient wind is available. Furthermore a long contact area between the wastewater and wind is necessary.

Although some of the oxygen required to keep the upper layers aerobic comes from reaeration through the surface, most of it is supplied by the photosynthetic activity of the algae which grow naturally in the pond where considerable quantities of both nutrients and incident light energy are available. Indeed, so profuse is the growth of algae that the pond contents are bright green in color. The pond bacteria utilize this ‘algal’ oxygen to oxidize the organic waste matter. One of the major end- products of bacterial metabolism is carbon dioxide which is readily utilized by the algae during photosynthesis since their demand for it exceeds its supply from the atmosphere. Thus there is an association of mutual benefit (‘symbiosis’) between the algae and bacteria in the pond.

The various components of waste stabilization pond treatment system are expected to be sump well, bar screen, grit chamber, anaerobic ponds, facultative ponds and sludge drying beds. The design criteria of each of these components are given below.

(a) Sump Well

a. Detention time:≤ 20 min
b. Number of units:≥ 2
c. Velocity of flow in the rising main:0.6 – 1.0 m/s
d. Detention time:≤ 20 min
e. Number of units:≥ 2
f. Velocity of flow in the rising main:0.6 –1.0 m/s
g. Velocity of flow in suction pipe 1.0 -1,5 m/s

(b)	**Bar Screen**		
	Minimum size of particles to be removed:		25 mm
	Inclination of bars with horizontal:		30 - 70^{o}

	Quantity of screenings production:		0.0015 – 0.015 m^3/ML of sewage flow
	Approach velocity of flow:		≥ 0.3 m/s at average flow
	Number of units:		≥ 2
	Size of bars:		10 mm × 50 mm
	Clear spacing between bars:		25 – 50 mm
	Maximum head loss through screen:	30 cm	
(c)	**Grit Chamber**		
	Detention time:	30-90 sec	
	Number of units:		≥ 2
	Velocity of flow:	0.15-0.4 m/s	
	Size of smallest particle to be removed: 0.2 mm		
	Specific gravity of particle:		2.65
	Design temperature:		20^oC
	Liquid depth:		1-2 m
	Length:	3-25 m	
	Freeboard:		0.3-0.5 m
	Quantity of grits:		0.02-0.10 m^3/1000 m^3 of flow
(d)	**Anaerobic Pond**		
	Depth:		2.5-5.0 m
	Detention time:	5-50 day	
	Organic loading:		400-4000 kg/ha/day
	BOD_5 removal efficiency:		50-80%
	L/B ratio:		2-3
	Embankment slope:		1(V):2(H)-1(V):3(H)
	Freeboard:		0.5-1.0 m

(e) Facultative Pond

Depth:	1.0-3.0 m
Detention time:	5-30 day
Organic loading:	40-400 kg/ha/day

BOD_5 removal efficiency:	70-95%
L/B ratio:	2-3

Embankment slope:	1(V):2(H)-1(V):3(H)
Freeboard:	0.5-1.0 m
(f) Sludge Drying Bed	
Depth of sludge application:	0.2-0.5 m
Sludge accumulation rate:	0.04-0.07 m^3/capita/year

Sludge drying time including cleaning: 15-30 days

Rainy period in a year: 5 months
No. of cycles/year: 7

Maximum size of one bed: 30 m × 8 m (240 m2)
Bed area: 0.25-0.10 m2/capita

8.3.3 Design Criteria of Reed Bed Treatment System

The Reed Bed wastewater treatment system is an innovative way of treating household wastewater or sewerage using concept of constructed wetlands. The function of reed bed treatment system is to reduce the BOD and the pathogenic organisms. The purification is achieved by sedimentation in the settlement tank and adsorbing of the organic and inorganic matters by reed and aerobic decomposition. The effluent from the reed bed treatment plant will have sufficiently less suspended particles and BOD and can be safely discharged into the nearby water course.

A typical wastewater treatment unit using constructed wetland technology with reeds comprises of a sedimentation/septic tank where the wastewater is collected and retained for 12 to 24 hours. Then, the wastewater is made to flow through, horizontally and vertically, on a bed of uniformly graded sand or gravel with reeds growing on it. As the wastewater flows through the bed of sand with reeds, it gets treated through the natural processes of filtration and microbial degradation. The reeds assist in the cleaning process by transporting oxygen to the microorganisms in the bed through its root hairs and by taking up some nutrients and other substances. It has been observed that the operational efficiency of constructed wetlands using reeds for total suspended solids (TSS), BOD, COD and ammonia (NH4) has been over 90 percent.

The reed bed treatment system consists of preliminary treatment of bar screen, grit chamber, primary treatment of settling tank and secondary treatment of horizontal flow bed and vertical flow bed filters connected in series. The effluent from the vertical flow bed is to be discharged to nearby water bodies/river. The design criteria of bar screen and grit chamber are same as waste stabilization pond treatment. The design criteria of the other components of the treatment system are given below.

(a) Septic Tank	
Detention time:	0.5-1.0 day
Retention time of sludge:	120 – 365 days
Sludge production rate:	0.035 – 0.04 m^3/person/year
Length/Width ratio:	2
Depth: BOD_5 removal efficiency: **(b) Horizontal Flow and Vertical Flow Reed Beds**	20-30% ≥1.0 m
Freeboard:	0.3-0.5 m

Depth of the horizontal bed:	0.3	– 0.6 m
Depth of vertical bed:	0.5	– 1 m
Bed slope:	0.5	– 1%

Media type:	Sand and Gravel
Porosity of media:	Around 40%
Reaction rate constant at 20 °C (K_{20}):	1.1 d^{-1}

8.3.4 Sludge Disposal

The WWSP will generate sludge over time, which will require periodic de-sludging. This is useful by-product and can be sold to farmers directly as field fertilizer. The municipality will periodically engage contractors to extract the sludge and transport it to the solid waste processing center for mixing with compost. Sludge can also be disposed at the municipal dumping site with permission from municipal authorities.

8.4 Excreta Disposal

8.4.1 Household Latrine

Over the years, various options for on-site sanitation units have been developed in developing countries through research and development. Literature on such experiences have been published and are available from agencies like UNICEF, WHO and INGOs like OXFAM. Some of the available options that can be considered and promoted through the Project are discussed in the sections below.

Although there are no clear definitions of a hygienic latrine, based on available literature and knowledge it can be referred to as a latrine with:

- Septic Tanks
- Water-pouring latrines
- Pour-flush
- Two-part (vault) latrines
- Improved and fully covered latrines (dug or pit latrines)

Features of such latrines and their potential disadvantages should be clearly developed. This shall help the concerned stakeholders in being able to make the right choice depending upon the physical characteristics, investment and people's preferences and habits.

The choice of technology depends largely on the topography and socio-economic conditions including affordability and general habits of the community members. Therefore, instead of being prescriptive, a range of choices needs to be developed for the consumers clearly listing the advantages and disadvantages of the different options. For peri-urban areas and higher income groups, pour-flush latrines with septic tanks can be the preferred option. In areas with low groundwater and low permeable soil, improved pit latrines properly covered can be recommended. For rural areas where the groundwater is likely to be contaminated due to the higher permeability of the soil, two- part (vault) latrines or even pour-flush latrines can be considered. However, the focus should be on behavioral changes, i.e. hygienic use of constructed latrines rather than the latest technology. Indifference to people's habits will only lead to unsustainable use of latrines.

8.4.2 School and other Public Latrines

The UWSSP envisages implementation of institutional and public latrines in the project areas where the water supply systems are to be built. Institutions like schools, kindergartens, and health stations can qualify to receive a grant of 85 percent of the latrine cost if the concerned authorities contribute 15 percent of the total cost. Similarly, latrines at public places like markets, bus stations, etc. can also qualify for a similar grant, if the responsible authorities agree to come up with the matching fund.

The type of latrine or technology used can be discussed with the Project and an appropriate choice based on cost, usage and need for the institution should be made. Experts under various projects have made the following recommendations for school latrines.

1- 300 pupils	=	5 latrines (3 compartments for girls and 2 for boys)
301- 600 pupils	=	6 latrines
601- 900 pupils	=	8 latrines
901 and above	=	10 latrines
Hand washing facility (at latrines)	=	4 taps for 300 pupils
	=	6 taps for 300 – 600 pupils
	=	8 taps for 600 – 900 pupils
	=	10 taps for above 900 pupils

If a school has two sessions, the highest number of pupils in either session should be taken into for design calculations.

Similarly, public latrines are desirable at market places and bus stops for the convenience of the public at large. Designs for such latrines should be carried out considering the intensity and frequency of usage by passengers or consumers. The focus be on the provision of adequate number of urinals and a limited number of pans / WCs. Hand washing provision should also be made for proper cleansing after use of such facilities. All public and institutional latrines should be gender friendly with provisions for incineration in girl's compartment for effective disposal for waste during menstrual periods. Similarly, latrines MUST have provisions for the access of differently- abled people. In this regard, type designs developed by UNICEF and other entities can be referred.

8.4.3 Design Criteria of Pit for Latrines

Wherever pit latrines are provided, the pits are used to collect the sewage including the night soil and urine. The sewage is digested in the pit. The liquid portion infiltrates into the surrounding soil while the gas escapes into the atmosphere. The remaining digested sludge accumulates in the pit. When pit is filled up within the 0.5 meters from the ground, the pit is filled with soil and the new pit is used. The ventilation of the pit will be necessary to prevent foul smell.

Volume of the pit (V) = N R T

Where

V = Volume of the pit in m^3

N = Number of users

R = Sludge accumulation rate in m^3/person/year (0.04-0.05

m^3/person/year) T = Filling time of pit in years (normally 2 years)

The shape of the pit should be circular. The depth of the pit should be increased by 0.5 meter as a provision of free board. The depth of the pit should be chosen in such a way that the ground water table is well below the bottom of the pit and the pollution of the groundwater is avoided.

8.4.4 Design Criteria of Septic Tank

In the towns and communities with sufficient water supply the disposal of sewage can be done by septic tank and soak pit. The sewage from the toilets are flushed into the septic tank. A septic tank is a combined sedimentation and digestion tank where sewage is detained normally for one day. During this period, the suspended solids settle down to the bottom of the tank. This is accomplished by anaerobic digestion of settled solids resulting in the reduction of the sludge volume, reduction in the biodegradable organic matter and release of gases like carbon dioxide, methane and hydrogen sulphide. The tank should be airtight with covered at top. The effluent from the septic tank although clarified to a large extent, is still foul in nature. Containing considerable amount of dissolved and putrescible organic solids and pathogens. Therefore, the septic tank effluent disposal needs careful consideration. The digested sludge from the septic tank needs to be cleaned once in 6 months to 3 years. The design criteria of septic tank is as follows.

Detention time (T) = 1 day

Volume of septic tank (V) = V1 + V2 + V3

Where,

V1 = Volume of sewage retention for sedimentation (m^3) = Q T

Q = Sewage design flow (m^3/d)

V2 = Volume for digestion of settled sludge (m^3) = 0.0425 N

N = Number of users

V3 = Volume for storage of digested sludge (m^3)

V3for various sludge cleaning period is given in the Table 29.

Table 29: Volume for Storage of Digested Sludge

Sludge Cleaning Period	V_3 (m^3)
6 months	0.0283 N
1 year	0.0490 N
2 years	0.0708 N
3 years	0.0850 N

Shape = Rectangular

Length/Width ratio = 2 to 4

Number of compartments = 2

Length of first compartment = 2 times length of second compartment

Minimum liquid depth = 1 m

Free board = 0.5 to 1 m

Design Criteria of Soak Pit

The septic tank effluent is malodourous, containing sizeable portion of dissolved organic content and pathogenic organisms, and hence it should be disposed of carefully so as to avoid any nuisance or risk to the public health. The septic effluent is usually disposed of by soil absorption method where effluent is absorbed in the surrounding soil. The soak pit is the most common method of septic tank effluent disposal. Soak pits are circular pit dug on the ground where septic tank effluent is disposed of. The septic tank effluent is absorbed by the surrounding soil. Care must be taken so as to avoid the groundwater pollution. The soak pit is covered at the top. It may be lined or unlined. The capacity of the soak pit largely depends on the infiltration/absorption capacity of the soil. The design criteria of soak pit is as follows.

Shape = Circular

$$D = \frac{Q}{\pi HI}$$

Where,

D = Diameter of soak pit (m)

Q = Design discharge of septic tank effluent (liters/day)

H = Effective depth of pit below the invert of inlet pipe (m)

I = Infiltration capacity of the soil ($l/m^2/d$)

Minimum diameter of pit = 1 m

Maximum diameter of pit = 3 m

Free board or depth above invert of pipe = 0.5 m

Provide more number of pits if the diameter of the pit exceeds 3 m or as per the site condition. Clear distance between the pits should be at least 2 times the diameter of the largest pit.

The infiltration/absorption capacity of the soil should be determined by percolation test. The absorption capacity of various types of soils is shown in the Table 30.

Table 30 : Absorption Capacity of Soil

S.N	Type of Soil	Absorption Capacity (liters/m^2/day)
1	Compact clay	≤ 20
2	Medium clay	20-30
3	Sandy clay	30-70
4	Fine sand	70-140
5	Coarse sand	140-150

8.4.5 Faecal Sludge Management (FSM)

Faecal Sludge Management is a safe process of handling the overloaded faecal sludge from the septic tanks of the dwellings/cluster and discharging the end products to natural drainage after treating it to safe and acceptable effluent standard and end products.

Faecal sludge quantity determination:

a. Sludge Production Method – based on the population, sludge generation estimation and the optimization of the resources for the effective implementation of the business plan.
b. Sludge Collection Method – based on the available resources and optimization of the resources for the effective implementation of the business plan.

Faecal Sludge Treatment Process: Based on the characteristics of the faecal sludge, the climatic conditions and the requirement of the nearby dwellings; the appropriate treatment sequences (aerobic / anaerobic) could be recommended to ascertain the effluent standard and end products.

9. DETAILED ENGINEERING DESIGN PROCEDURES

This section describes briefly the design requirements for the detailed engineering phase of a typical project. Detailed engineering design (DED) shall only commence when the Feasibility Study (FS) for a project has been completed and the project has been found feasible technically, socially and environmentally and the community has accepted the proposed alternative. Once clearance has been obtained to proceed for the detailed engineering design the following procedures should be adopted in sequence to provide enough details such that the construction of the selected scheme / project is effectively carried out.

Review of the Feasibility Study Report – Reconfirmation and reassessment of the source, demand and other major findings of the feasibility study shall form an integral part of the detailed engineering design. The proposed technical alternatives in the FS including the sources of water need to be carefully assessed. This is very essential to get a background of the proposed system prior to proceeding for the DED.

Verification of the Service Area Population and Growth Rate – The designated service area and its population needs to be revisited to verify the settlement pattern, existing population and its growth rate. This has a huge bearing on the cost optimization process during the detailed engineering design of the system. It needs to be verified if any pocket settlements are not included or left out because of any social, political or economic reasons. When it is not technically or financially feasible to be included on the piped scheme, they must be given the non-piped water option.

Source Verification - Re-measurement of the surface water source or verification of the yield of the groundwater source during the detailed engineering design phase (in the driest season) is necessary to increase the reliability of the already existing hydrological information database. This is also necessary to confirm, if there exist any source disputes or problems in borehole location etc.

Water Quality Assessment - Sampling to conduct water quality analysis of the proposed sources again during the DED is required to consolidate the findings of earlier water quality assessments. Additional sampling to monitor other water quality problems like arsenic and iron also needs to be carried out to assess the extent of such problems. In addition, if the source is a surface water system, pre and post Monsoon water quality assessment shall be required.

Preparation of Digital Base Map– A more recent development has been the development of a digital database for maps on GIS platforms. Base maps can be prepared either by digitizing or scanning existing topographical maps (or even aerial photographs) into vector formats from raster images. These digital base maps can be used in GIS platforms to develop a GIS for the water systems with various layers giving details about the service area, pipe network, house connections, topographical features, etc. This can also be a powerful tool for operating and managing the built system, as any additional or changes made to the system can be instantly updated in the GIS system for the water network.

Field Verification Survey - During the DED field survey for the ground truth information on the base maps needs to be carried out. These surveys also shall verify and consolidate the topographical features of the area under consideration. Adequate details are needed to establish the length of the pipes and elevation of the nodes in the system. In addition, other topographical details for areas where major system components like pump houses, water treatment units, reservoirs, water towers, etc. are to be located need to be collected.

Finalizing Design Criterion and Parameters – Design criteria associated with design period, water demand, supply hours (consumption pattern), pressure rating, water quality and treatment etc. needs to be adequately addressed and finalized before beginning the detailed design.

Water Demand – The total water demand for the service area including domestic, non-domestic (institutional, commercial, etc.) and unaccounted for water (UFW) for various project stages needs to be developed and checked with the scenarios developed in the Feasibility Report.

The Nodal demand at this stage should be based on the actual settlement pattern of the households and the actual numbers of households to be served from a node. A simple way of doing this is to have a household count of the service area and locate the number of households on the system layout map. Then the number of households with population can be lumped at various nodes of the system for actual demand assessment.

Hydraulic Design of System - Hydraulic engineering design for raw water system, treatment unit, and storage and distribution network needs to be carried out in greater detail. This should evolve into a balanced pipe network giving details of pressure and flow in the network. The minimum pipe velocity and minimum residual pressure at nodes should be maintained. The pipe network should give all relevant details on individual network junctions, location of flow gauging meters and other appurtenances, as designed.

Structural Design - Structural engineering design for civil structures like the intake, pump houses, treatment facilities, storage reservoirs, water towers, etc. needs to be done giving all relevant details for construction.

Detailing for Electro-mechanical Equipment - Detailed mechanical installation layouts for pumps, electrical supply, piping system at WTP and reservoir, etc. need to be carried out.

Detailing for SCADA Components - Detailed Line diagram for SCADA Project Components, Schematic diagram for SCADA based System , Arrangement diagram, Power supply, Coordinates, signal strength, site space Photograph etc. need to be carried out.

Rate Analyses - Realistic unit rates for individual cost components need to be developed using latest market rates and standard norms.

Detailed Estimation and Drawings - Detailed Bill of Quantities (BOQs) and drawings for all system units are required for effective implementation.

Bid Document Preparation - The Standard Bidding Documents (SBD) needs to be followed to develop Project specific standard bidding documents. The bid documents will be prepared in the following four volumes:

Volume I : Invitation / Instruction to Bidders, Bid Data Sheet, Evaluation and Qualification Criteria, Sample Bidding Forms

Volume II: Conditions of Contract (General and Special) with Contract Data and Bill of Quantities

Volume III: Technical Specifications

Volume IV: Drawings.

10. OPERATION AND MANAGEMENT ISSUES

Another critical issue in determining the type of water system for a service area is the ease in the operation and maintenance of the built system. Higher operating or recurring costs mean higher tariff. Operation and maintenance costs of water systems constitute three major components: personnel or human resources costs, power or energy costs and regular maintenance expenditure like use of chemicals etc. These factors need to be kept in mind while designing the water system such that the operating expenses are low. Power consumption is the single biggest factor in the recurrent costs of water systems, as most of them involve pumping. Therefore, it is necessary that pumping be introduced only when it is most necessary. Secondly, the selection of a pump should be done critically so that the chosen pump consumes less power and operates at a greater efficiency.

11. DRAWINGS

Once the design of a scheme is completed, the following drawings should be attached as a part of the report wherever applicable

1. The layout plan of the all the proposed water supply systems from the source to the service areas. It should show all the components mentioned above as they exist in the design.
2. Schematic flow diagrams of all the proposed systems
3. Detailed site plan with contours of all the proposed components such as intakes, reservoirs, water treatment plant etc.
4. Hydraulic profile of the water treatment plant
5. Pipeline longitudinal profile showing the following details
   - Chainage
   - Ground elevation
   - Nature of soil/terrain along the alignment.
   - Hydraulic grade line, static line, residual head.
   - Pipe type (materials) and diameter, pressure.
   - Flow velocity, and flow in each section.

Location of proposed components the scale of drawings shall be as follows.

Drawing	**Scale**
Contour map	1:100
Layout plan	1:5000
Pipeline longitudinal profile	1:2000 Horizontal; Vertical1: 100
For all water supply components	
Plan & Details with contour	1:100
Details	1:10 to 1:30
Miscellaneous Drawings	1:10 to 1:30

12. FINANCIAL AND ECONOMIC ANALYSES

12.1 Methodology

This is simply methodological applications of principles of economic analysis for the Second Small Town Water Supply and Sanitation Projects; it is also a practical guide that is suggested to be used by economists in the Financial and Economic analysis of the projects.

<u>Initial Capital Cost Breakdown</u>

The initial cost is to be breakdown into foreign and local cost. The local cost is divided into skilled labor, unskilled labor and local materials.

Operation and Maintenance Cost

Personnel Cost – As per requirement & logical justification

i. Manager
ii. Technician
iii. Accountant
iv. Administrator
v. Cleaner
vi. Guard
vii. Office boy
viii. Others if any with logistical justification

Annual O&M Cost

ix. Spare parts and maintenance cost with logical justification
x. Electricity cost based on NEA tariff with annual increment of 3%
xi. Office expenses / Stationery
xii. Include Diesel Cost (10% of the Pump Operation cost)
xiii. Others If any with logistical justification

Replacement Cost

Machinery and Equipment Cost

xiv. 10% of the Machinery and Equipment Cost (after 10 years)
xv. Meter replacement cost after 7 years
xvi. Other costs if any with logistical justification

Water Demand Analysis

Population of the project area:

- % of population served during the project period
- Increment of the population served up to the end of the project

- Total population served up to the end year

Fully Plumbed:

- % of population served
- Per capita consumption (lpcd) – constant over the project period
- % Increase of population served during the project period

Yard Tap:

- % of population served
- Per capita consumption (lpcd) – constant over the project period
- % Decrease of population served during the project period

Stand posts:

- % of population served
- Per capita consumption (lpcd) – constant over the project period
- % Decrease of population served

Commercial Enterprises:

- Number of existing enterprises
- Water demand per enterprise - optimum but logical
- Assume constant over the project period

Institutional:

- Number of institutions
- Number of students (Day Scholars)
- Per capita consumption (lpcd) – constant over the project period
- Number of students (Boarder)

Industrial:

- Number of Industries
- Water demand per industry – optimum but logical
- Assume constant over the project period

Leakage and Wastewater:

- Leakage and wastewater assumed at 10% of the Domestic Demand
- Fire demand: 1% of the total demand

Total water demand:

The total water demand is estimated including domestic, commercial, institutional, non-domestic, leakage and wastage and fire demand etc.

Tariff Structure

Tariff Setting and Cost Recovery: The Project's financial sustainability depends on the appropriate tariff level to recover the necessary capital during the project period such as O&M costs, rehabilitation and recurrent costs as well as the repayment of the TDF loan. It is expected accumulate cash in hand a portion of initial capital cost at the end of cash flow analysis for further development and expansion in future. However, the tariff should determine at lowest maintaining the sustainability and within the affordability and willingness to pay of the lower income households.

The incremental blocked tariff (IBT) structure implies cross subsidy from better off, higher volume consumers to the poor, minimal volume consumers. The tariff will be proposed on the basis of the incremental volumetric block system to increase the revenue as well as to discourage the wastage of water.

The incremental block tariff system is also justified on socio-economic grounds so that the lower income group or poor community will be in an advantage. However, the tariff will be fixed considering socio-economic, financial and economic viability of the project for efficient utilization.

The IBT system can also be applied in various locations and communities such as urban and semi-urban areas also if it is crucial for cross subsidy.

The tariff set up may be a tool for demand management; the increase in tariff may cause a decrease in water demand and vice versa. The tariff structure needs to be based on Incremental Blocked Tariff (IBT) system as follows:

i. up to 10 cum - "" NRs xx.xx Rate per cum (basic rate)
ii. 11 to 20 cum - (NRs xx.xx times 1.3 basic rate, 30% increase)
iii. 21 cum & higher - (NRs xx.xx times 1.7 basic rate, 70% increase)

Minimum Consumption Level: The minimum water consumption of household per month is considered at 10 cum and calculated the monthly bill at the tariff rate.

Note: Analysis should be based on the information available in the socio-economic survey results

Financial Sustainability

The financial sustainability of the water supply and sanitation project is measured in terms of the financial rate of return (FIRR) that covers the investment, operation and maintenance and rehabilitation, replacement and administration cost as well as the repayment of loan for the construction of the project. In general, the FIRR should be greater than the overall Weighted Average Cost of Capital (WACC), which is tentatively estimated at 4.0%.

Cost Recovery

The accumulated cumulative amount at the end of the cash flow period of revenue generation a portion of initial investment cost for development and expansion in future; especially for Urban water supply and sanitation projects, because such types of projects are not profit oriented rather service oriented and it is essentially the duty of the government to provide services to the people.

The revenue from tariff should also cover the annual costs of operation and maintenance, rehabilitation, replacement and administration for the long term sustainability of the water supply and sanitation project.

Repayment of Loan

The repayment of the loan is one of major factors of sustainability and acceptability of the project. The interest amount on grace period of 5 years is accumulated to loan amount to make total loan for annuity calculation. The loan is repaid annually amortizes for the 15 years (annuity payment) on loan amount plus accumulating the interest amount on grace years period. The annuity payment is paid annually for the period of 15 years after the grace period of 5 years. For ensuring the repayment of the loan, at least the cumulative cash flow in the financial cash flow statement must be positive so that loan repayment is continued without being intermittent.

Affordability

The monthly bill should be within the affordability of the household (HH). The monthly bill payment must be within 3% - 5% of the HH income Level of the WUSC.

Willingness to Pay

- House Connection

Willingness to pay of the beneficiaries or (WUSC) need to be confirmed from the household survey for the connection cost of fully plumbed taps.

- Monthly Water Bill

Similarly, beneficiaries need to show their willingness to pay for monthly water bills for the sustainability of the project. The willingness to pay for water is based on the reliability, quality and continuity of water supply.

Sewerage and Sanitation

Public sanitation facilities, which include public toilets, sludge drying beds, storm water drainage, and wastewater management will be financed jointly by WUSCs and local governments 15% of the total cost and 85% will be subsidized by the Government. Out of the 15% of the total cost of Sanitation and Sewerage System, the percentage share may be set under the mutual understanding between the local government and WUSC. On-site sanitation will be the responsibility of individual households, but grants through the OBA (Output Based Approach) approach will be provided to households to facilitate construction of private latrines. All other related costs including consulting services, NGO services, training, capacity building, and incremental administrative expenses will be financed by the Government using Project funds. O&M costs for water supply will be fully recovered by WUSCs, while those for sanitation services (sludge management, off-site sanitation, and drainage) will be paid for by WUSCs and local governments through a mutual agreement.2

The part of O&M cost for the sanitation services will be responsible by WUSC as per the mutual agreement with the local government. The Project's financial sustainability assumes an appropriate tariff level to recover at least necessary capital and O&M costs in the water supply subcomponent. Hence, it is essential to determine tariff rate of water supply sub component project to cover the O&M cost of sanitation services also.

The economic and financial analysis will be performed using the water supply component only. However, the lump sum amount of operation and maintenance cost of sewerage and sanitation needs to be included in the operation and maintenance cost of water supply sub component for financial analysis to determine the tariff rate for the sustainable of sanitation and sewerage sub component also. . With the inclusive of O&M of sanitation services the tariff rate will be slightly increased but there will be no any hassle for WUSC to handle O&M cost for sanitation service.

12.2 Economic Analysis

Assumptions

- Economic Opportunity Cost of Capital (EOCC) is considered at 12%
- Standard Conversion Factor (SCF) is considered at 90%
- Shadow Wage Rate (SWR) is considered at 70%
- All costs, in particular imported tradable inputs are exclusive of duties and taxes for economic analysis because they represent transfer payments

- The physical contingencies are excluded because they represent real consumption of resources
- For the analysis the economic opportunity cost of raw water is assumed to be zero. It is assumed that raw water is not diverted from any other commercial use such as irrigation.

Economic Benefits

- Improvement in Health (Expenditure of HH on health due to water borne diseases and poor sanitation)

The expected impact of the Project is improved health and economic and environmental improved, affordable, and sustainable water supply and sanitation services which are governed and managed by locally accountable representative bodies The Project is expected to provide a high level of water supply services to about 240,000 people in about 20 small towns.

Sanitation services such as on-site sanitation (private latrines), public toilets, wastewater management facilities (if justified), and storm water drainage will also be provided in the same towns through an integrated approach. About 270,000 people will have access to and use improved sanitation facilities. Supplemented by health and hygiene education programs in these towns, the Project will bring significant health benefits, to be measured by the reduction in the occurrence of waterborne diseases. Improvement in the Environment (Tangible and Intangible Benefit)

The benefits of improved sewerage or sanitation due to an environment will better living conditions and bring a reduction in public health risks in the Project area thereby increasing the Quality of Life. This will be achieved through the effective removal of wastewater from in and around living areas and prevention of wastewater from entering drains and rivers and, in some areas, wreckage water supply pipelines. Improved disposal of wastewater will result also in more pleasant surroundings through a reduction in odor and an improvement in the aesthetic quality of drains, rivers and low-lying areas. However, environmental benefits will be difficult to quantify.

Tangible and Quantifiable Benefit

Three main quantifiable economic benefits of the project will be considered in the analysis. Time saving for water collection, in particular, will be a major benefit expected from the Project, mainly for women and children. On the with-project situation, the economic benefits associated with the resource costs saving in terms of time saving to collect water, incremental benefits in terms of increased water demand as additional consumption, unaccounted-for-water in terms of utility of non-revenue water will be analyzed. These benefits are considered as economic benefits for the analysis of the economic viability of the project.

Time saving in water collection

Resource cost saving in terms of time saving: The time saved in collecting water for the community households due to the access of water supply is valued at the rate of shadow price of labor (unskilled labor) which is considered as an intangible benefit to the society and hence considered as an economic benefit.

Incremental Benefit (Surplus Demand)

Surplus Demand – Incremental Sale: The surplus demand due to the access of the water supply project (assumed supply minus the present consumption level) is valued at the rate of the weighted average of the willingness to pay (WTP) and the value of time saving of the community; which is also considered as economic benefit.

Customer Contribution

The customer contribution is also considered as an investment as well as a benefit to the "with project" situation since it is not directly the project cost but the capital cost including the cost of connection and meter. Hence, the customer contribution is considered as an investment as well as a benefit.

Non-revenue Water Demand

Non-revenue Water –The wastage of water assumed at 10% is considered to be consumed partly by the community: The loss of water is considered in the project at 10% of the total demand but in practice 25% of wasted water is assumed to be utilized by the community as non-revenue water and is valued at the weighted average tariff of incremental and non-incremental water which is also considered as an economic benefit.

Average Incremental Economic Cost (AIEC)

AIEC is the discounted value of the incremental capital costs and operating costs (in economic prices) over a 20-year period divided by the discounted value of the incremental quantity of water sales as a result of the proposed investment. The discount rate used is the social opportunity cost of capital (SOCC) estimated at 12%.

Economic Internal Rate of Return (EIRR)

EIRR is calculated as the rate of discount for which the present value of the net benefit stream becomes zero, or at which the present value of the benefit stream is equal to the present value of the cost stream. For a project to be acceptable EIRR should be greater than the economic opportunity cost of the capital.

Net Present Value of the Net Present Benefit

The results from the analysis are expected to show that the project is economically viable when the economic net present value of the net benefits is positive. The positive and higher economic net present value of the net benefit will be considered as viable of project in economic perspective.

Poverty Impact Ratio

Economic benefits to the real poor are associated with consumer utility and labor supply compared with the economic benefits of the whole project. The gains and losses are determined by the difference between the economic and financial benefit and costs. The actual net benefit accruing by real poor or termed as poverty impact ration (PIR) is calculated based on the proportion of net economic benefit by the real poor with the economic benefit of the project as a whole.

The cost and benefits of a water supply project (WSP) are shared among different groups. Based on the results from the financial and economic benefit-cost analyses, an assessment of the distribution of the project benefits and costs can be given to show which participant will gain from the project or incur a loss.

PIR is expected to be higher than the ratio of the existing core poor in the community or the small town.

13. ENVIRONMENTAL AND SOCIAL SAFEGUARDS

13.1 Purpose

All subprojects will be assessed prior to program implementation, using the following procedures. The environmental impacts of the small towns/urban subprojects will be assessed by the Executing Agency as required by ADB policy and the national laws.

This document provides the structure through which such assessments will be conducted. The document complies with ADB's Safeguard Policy Statement (SPS 2009), and the GON Environmental Protection Act (1997) and Environmental Protection Rules (1997, amended 2007).

13.2 Overview of Type of Subprojects to be assessed

The subprojects will consist of water supply, sewerage, drainage and sanitation schemes as these are the most urgent priorities in most emerging towns in Nepal. Table 31 shows the main components of such schemes. The impacts will always need to be examined by a process of environmental assessment (involving an IEE or EIA as appropriate), because the nature and significance of an impact can change with location and the specific details of the project.

Table 31: Possible Elements of the Sub-projects

Sub-project Type	Major Components (new or renovated)	Possible Main Elements (new or renovated)
Water supply	Water intake	Deep tube well/borehole well/Sump well Ground reservoir tank Surface water intake Overhead storage tank Pump house and pumps
	Treatment unit/ Treatment plant	Roughing filter Sedimentation tanks Slow sand filters Rapid sand filters Pressure filters Iron removal unit Chlorination unit Staff house Laboratory (Small
	Water transmission	Water transmission main Ground reservoirs Overhead reservoirs Pump house and pumps
	Distribution network	Distribution main Local distribution network House Connections Household meters
Drainage/Storm water works	Network	Drains Rain inlets Manholes

Sub-project Type	Major Components (new or renovated)	Possible Main Elements (new or renovated)
		Outfall River bank protection
Sanitation/Sewage Management	Sewage network	Sewers and manholes Reed bed/Treatment Plant Sludge drying bed
	Public Toilet	Toilet Septic tank and soak pits Sludge drying bed
Solid waste management	Waste processing center	Covered concrete shed

13.3 ADB Environment Policy

Environmental Classification

ADB uses a classification system to reflect the significance of a project's potential environmental impacts. A project's category is determined by the category of its most environmentally sensitive component, including direct, indirect, cumulative, and induced impacts in the project's area of influence. Each proposed project is scrutinized as to its type, location, scale, and sensitivity and the magnitude of its potential environmental impacts. Projects are assigned to one of the following four categories:

(i) **Category A.** A proposed project is classified as category A if it is likely to have significant adverse environmental impacts that are irreversible, diverse, or unprecedented. These impacts may affect an area larger than the sites or facilities subject to physical works. An environmental impact assessment is required.

(ii) **Category B.** A proposed project is classified as category B if its potential adverse environmental impacts are less adverse than those of category A projects. These impacts are site-specific, few if any of them are irreversible, and in most cases mitigation measures can be designed more readily than for category A projects. An initial environmental examination is required.

(iii) **Category C.** A proposed project is classified as category C if it is likely to have minimal or no adverse environmental impacts. No environmental assessment is required although environmental implications need to be reviewed.

(iv) **Category FI.** A proposed project is classified as category FI if it involves investment of ADB funds to or through a FI.

13.4 Preparation of Initial Environmental Examinations (IEEs)

An IEE describes the studies conducted to identify the potential environmental impacts of a proposed development, and is prepared when impacts are unlikely to be highly significant and can be mitigated relatively easily. In this program ADB policy requires thatan IEE (or EIA) is conducted for each subproject. Impacts are screened using ADB's Rapid

Environmental Assessment Checklists. For small town water supply and sanitation projects the IEEs are usually adequate.

Any new subprojects must comply with national legislation and procedures and ADB policy. If the criteria shown in the environmental criteria table in the next chapter are followed in the selection and development of new subprojects, most of them shall have relatively minor environmental impacts and the procedure for environmental assessment should then be straightforward and shall follow the normal IEE procedure described earlier. EIA will be implemented if either the Government's or ADB's rules/policy so require.

Appendix 2 and Appendix 3 give the format for preparing the TOR and IEE. It is suggested that ADB's SPS (2009) (Safeguard Policy Statement) and Environmental Assessment and Review Framework/Policy (EARF/EARP) for Emerging Towns Sub-Projects be used as references.

13.5 Gender Equity and Social Inclusion – Action Plan Implementation (GESI- AP):

- WUSCs formed with at least 33% women members and at least one woman in a key post
- Representation from socially excluded and vulnerable groups in WUSC ensured
- Awareness campaign - (50% Women)
- Smart utility management and leadership of stakeholders (33% Women)
- Participate in project orientation and consultation sessions by local communities (33% women and 25% socially excluded and vulnerable groups)
- Output based Aid (OBA) subsidized for piped water connection and toilet facilities to 100 % poor and vulnerable households including WHHs in project coverage areas

ANNEXES

Annex 1 – EVALUATION PIPE MATERIALS FOR TRANSMISSION AND BULK SUPPLY

1 BDS Pipeline Material Introduction

For large diameter pipes used in water supply conveyance and distribution, a number of materials exist including Ductile Iron, Steel, PVC, Polyethylene, Concrete, and Glass Reinforced Plastic (GRP), among many others. For the current study, it seemed prudent that a brief comparative assessment of major pipe materials for large diameter pipes be considered before recommending a particular material for subsequent use and implementation. The following comparison is for the most appropriate materials for large diameter pipes to be considered for the Bulk Distribution System (BDS) or transmission mains; PE, Steel and DI. These materials are compared for their relative ease in transportation, installation and laying, material and laying costs, availability of fittings, internal and external coating and operation and maintenance.

A. Ductile Iron (DI) Pipes

Ductile Iron pipes have been extensively used for water supply throughout the world because of the flexibility and ease in installation, pressure ratings, longevity and resistance of different soil conditions and strength. The most common type of DI pipes are conforming to ISO 2531 (2009) and EN 545 (2006). The principal properties of the ductile iron is a yield stress of over 300 MPa, tensile strength of over 420 MPa, elongation of over 10%, and Brinell hardness less than 230. The impact strength of DI pipes is exceptional and allows a variation in pipe alignment up to 4%; depending upon pipe diameter. Normally, DI pipes are available for diameters ranging from 80 mm to 1200 mm, but larger diameter DI pipes up to 1800 mm are also available. Normal allowable operating pressure for DI pipes range from 25 to 40 kg/cm2. The internal surface of DI pipes normally has a cement mortar lining, which is applied by a centrifugal process. The cement mortar lining shall be in accordance with ISO 4179 – 2005 with minimum thickness ranging from 2.5 to 7mm depending upon the pipe diameter. Similarly, the external protective coating of DI pipes shall be zinc plus bitumen. The metallic zinc coating shall be in accordance with ISO 8179 Part 1 – 2004. In addition, a bituminous varnish is applied over the zinc coating, also in accordance with ISO 8179 Part 1 – 2004. This provides a high degree of protection against most soil conditions. For aggressive soils the pipes can be protected by a coating of polyurethane. The water tightness of DI pipes will be ensured by push-on joints with gaskets made of elastomer rubber (EPDM Ethylene Propylene Diene Monomer or SSBR Styrene Butadiene Rubber) conforming to ISO 4633 – 2002.

B. Polyethylene (PE) Pipes

Polyethylene pipes are extensively used for bulk water supply especially in Europe considering their flexibility and ease in laying quickly. PE pipes are designed, manufactured and supplied under a BS EN ISO 9001:2000 accredited Quality Management System. Two most common types of PE pipes used are PE80 and PE 100. PE80 – This is the term used to denote the polyethylene material which has been widely used for gas, water and industrial applications for many years. The terms MDPE and HDPE were commonly used to describe this material.

PE100 – This is a term used to denote high performance polyethylene. PE100 is a higher performance material than PE80 and demonstrates exceptional resistance to rapid crack propagation as well as to long-term stress cracking. Moreover, the higher strength of PE100 permits thinner pipe walls than PE80 for the same operating pressure. PE100 uses less polymer and provides for a larger bore and increased flow capacity for a given nominal pipe size. This can result in significant cost savings at certain sizes and pressure ratings.

PE pipes are manufactured from a diameter of 20 mm to 1000mm normally. However, larger diameter PE pipes can also be manufactured subject to scale or magnitude of order. Allowable working pressure for PE80 pipes varies from 12.5 to 8 bars, whereas for PE100 it varies from 16, 10, 8 and 6 bars for SDRs (ratio of outer diameter to average wall thickness) 11, 17, 21 and 26.

C. Mild Steel Pipes (MS)

Mild Steel Spiral Weld pipes have the advantage that they can be designed for any pressure and this can result in cost savings. The pipes can be designed to withstand considerable longitudinal forces eliminating the need for anchor (thrust) blocks in many locations. The joints are generally welded giving little tolerance for ground movement or for making on-site adjustment to the pipeline alignment during installation. Mild Steel Spiral Weld pipes need to conform to IS 3589 / 2001.

Cement mortar linings are effective in preventing internal corrosion and also will maintain their surface conditions over their lifetime (unless particularly aggressive waters are being conveyed - which is not expected.). Cement mortar linings can be works applied or applied in situ. Works application is carried out by spinning requiring special equipment and results in pipes being heavy, difficult to handle and the lining

is prone to damage during transportation to site. Alternatively, buried steel pipeline can be coated internally, with a single coat two part solvent free high build liquid epoxy lining as per AWWA C210-07 suitable for potable water application and approved by

NSF International Standard NSF/ANSI-61 2004. Liquid epoxy lining for internal coating has many advantages over cement mortar lining.
The preferred external coating for buried steel pipeline is prefabricated polyolefin tape coating as per AWWA C 214-07. It is the general practice that special sections, miter bends, tees, connections, fittings in buried steel pipeline network should be coated externally, with prefabricated polyolefin tape coating as per AWWA C 209-00.

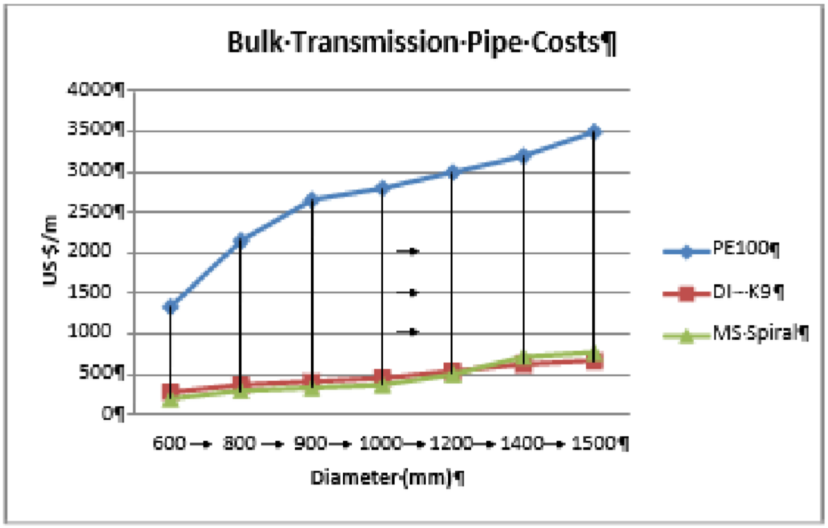
2 Comparative Assessment of Pipe Materials for BDS

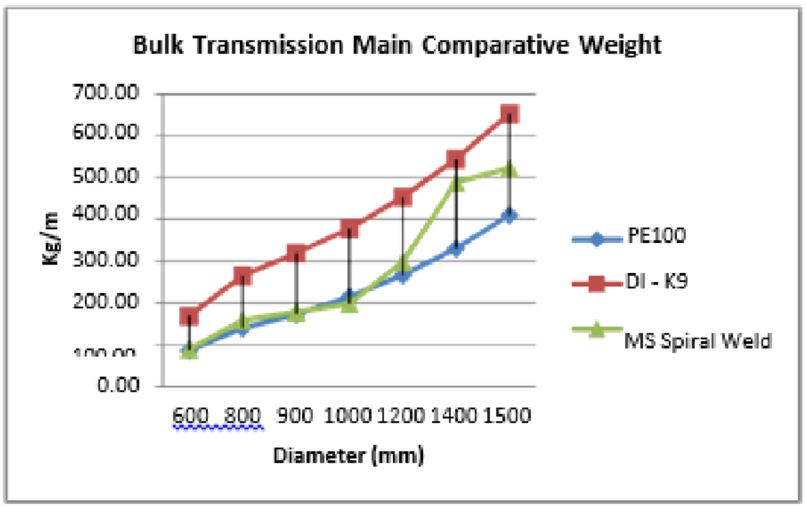
The three pipe materials discussed have their own distinct advantages and disadvantages. However, for the present purposes the pipe materials have been comparatively assessed in terms of the following factors – transportation; laying and joining; strength, operation and maintenance (including availability of spare parts) and initial investment costs. The comparative table below summarizes the advantages and disadvantages of the three pipe materials:

Pipe Material	Advantages	Disadvantages
PE100	• PE100 pipes are relatively lighter in unit weight and easier to transport. • The internal surface is much smoother resulting in higher "C" value or lower resistance. • Repair and maintenance is relatively easy • More suited for conveyance of potable water. • Greater laying flexibility and requires average bedding requirements.	• The pressure rating of the larger diameter pipes is limited. • To ensure the pipeline has good structural properties, a high standard of bedding is required. • Without good storage conditions plastic pipes deteriorate, and storage of materials for future repairs may be difficult. • Butt-fusion welding of joints needs special equipment and skill. • Cost for unit length for larger diameter pipes is extremely high.
DI	• Pipes are relatively strong and can withstand large external loads with modest bedding. • The majority of standard pipes and fittings can handle pressures up to 16 Bar. • The flexible joint system means that earthquake movements can be accommodated.	• Restrained joint can be provided by most manufactures where the pipes are to withstand some longitudinal forces; however, generally anchor (thrust) blocks are required. • Unit weight of pipes is generally higher than PE and MS Spiral Weld Pipes. • Transportation cost can be relatively

	• The strength of the pipes means they are less likely to be damaged than other materials. • Ductile iron pipes can be laid quicker with short lengths of trench causing less disruption to road traffic. • The system can be modified and additional facilities easily added to the pipe network. • Ductile iron pipes are semi-rigid pipes and generally do not require bedding to be of as high a standard as PE or MS pipes.	higher. • Per unit cost of DI pipes in comparison with MS Spiral Weld Pipes is similar, but becomes cheaper for larger diameter pipes.
MS Spiral Weld Pipes	• Steel pipes have the advantage that they can be designed for any pressure and this can result in cost savings. • The pipes can be designed to withstand considerable longitudinal forces eliminating the need for anchor (thrust) blocks in most cases. • Improved capacity and flow characteristics are in utilizing liquid Epoxy Coating, which gives higher "C" factor. • Liquid Epoxy coating offer very much reduced weight and have longer life	• The joints are generally welded giving little tolerance for ground movement or for making on-site adjustment to the pipeline alignment during installation. • Welding the joint in the trench will require a large excavation at each joint to allow access for the welder to make the joint. For the larger pipes it will take a day to make each joint. • The pipes require a good standard of bedding to ensure deflections are not excessive. • Addition of new sections of pipeline to an existing steel system requires specialist skills in welding. • Lined and coated pipes from manufacturers are usually transported in 40ft trailers to carry pipe lengths of 12 meters. • Because of the special need of MS Spiral Weld Pipes internal and external coatings especially at joints has to be carried out by specialists. • MS Spiral Welded Pipes' unit costs are similar to that of DI pipes, but are slightly costlier for larger diameter pipes.

A comparison of the three pipes with reference to their unit costs and weight is presented in the charts below. The unit costs have been obtained from various sources including the web. PE pipe rates for large diameters are high in comparison because they follow British and European standards.


Bulk·Transmission·Pipe·Costs¶
4000¶
3500¶
3000¶
2500¶
2000
1500
1000
500¶
0¶
US $/m
600 800 900 1000 1200 1400 1500¶
Diameter·(mm)¶
PE100¶
DI·-·K9¶
MS·Spiral¶


Bulk Transmission Main Comparative Weight
700.00
600.00
500.00
400.00
300.00
200.00
0.00
Kg/m
600 800 900 1000 1200 1400 1500
Diameter (mm)
PE100
DI - K9
MS Spiral Weld

3 BDS Pipe Material Recommendation

After considering all the issues related to construction, maintenance and investment of the three pipe materials discussed above, it has been decided that DI pipes conforming to ISO 2531 (2009) and EN 545 (1995) be used for all bulk transmission mains, ring and transversal mains including distribution pipes greater than 300 mm diameter considering their longevity, ease in construction and laying and relative investment requirement.

Evaluation of Pipe Materials for Distribution Network

4 Pipeline Material Introduction

For pipes used in water supply conveyance and distribution, a number of materials exist including Ductile Iron (DI), Galvanized Iron (GI), PVC, Polyethylene (PE) and Glass Reinforced Plastic (GRP), among many others. Several studies in the past have made specific recommendations based on the pipe size to be used and the soil conditions and experience of local craftsman in laying and joining distribution network pipes. Other considerations like cost ease in installation and better performance over a long period of time also needs to be assessed.

For the current study, it seemed prudent that a brief comparative assessment of major pipe materials for distribution network pipes be considered before recommending particular materials for subsequent use and implementation. The following comparison is for the most appropriate materials to be considered for the Distribution Network Improvement (DNI), i.e., DI, PE, GI, and uPVC. These materials are compared for their relative ease in transportation, installation and laying, material and laying costs, availability of fittings, internal and external coating and operation and long term maintenance.

A. Ductile Iron (DI) Pipes

Ductile Iron pipes have been extensively used for water works throughout the world because of the flexibility and ease in installation, pressure ratings, longevity and resistance to different soil conditions. The most common type of DI pipes are conforming to ISO 2531 (2009) and EN 545 (2006). The principal properties of the ductile iron is a yield stress of over 300 MPa, tensile strength of over 420 MPa, elongation of over 10%, and Brinell hardness less than 230. The impact strength of DI pipes is exceptional and allows a variation in pipe alignment up to 4%; depending upon pipe diameter. Normally, DI pipes are available for diameters ranging from 80 mm to 1200 mm. Normal allowable operating pressure for DI pipes range from 25 to

40 kg/cm2. The internal surface of DI pipes is coated with cement mortar lining, which

is applied by centrifugal process. Similarly, the external protective coating of DI pipes is zinc plus bitumen. This provides a high degree of protection against most soil conditions. Other features of DI pipes shall be similar to the earlier description made under BDS pipe material options.

B. Polyethylene (PE) Pipes

Polyethylene pipes are extensively used in Nepal for their flexibility and ease in laying. Internationally, PE pipes are designed, manufactured and supplied under a BS EN ISO

9001:2000 accredited Quality Management System. Two most common types of PE pipes used are PE80 and PE 100. However, in Nepal PE or HDPE, as it is more commonly known, is manufactured as per NS 40 – 2040 or relevant Indian Standards of IS 4984 – 1978. PE / HDPE pipes are generally manufactured in pipe diameters ranging from 20 – 110 mm under pressure ranges from 2.5 – 10 kg/cm2. However, HDPE pipes of larger diameters, i.e. 110 mm onwards are also manufactured, if required in Nepal.

C. Galvanized Iron Pipes (GI)

Galvanized Iron (GI) pipes are also extensively used in pipe distribution systems in Nepal. Although imported in the past, GI pipes are manufactured in the country conforming to NS 199 – 2046 or relevant Indian Standards of IS: 1239 (Part I) - 2004. GI pipes are normally available in pipe diameter range of 15 – 100mm, but can be manufactured for greater diameters of 125 and 150mm. There are three pressure classes under which GI pipes are manufactures – Light, Medium and Heavy. However, only Medium and Heavy classes of GI pipes are recommended for use in reticulated water systems with working pressure of 15 and 20 kg/cm^2.

D. Plasticized PVC Pipes

More recently uPVC pipes are being used in urban water systems, as they are available in large range of pipe diameters and pressure classes. Although there are a few manufacturers of uPVC pipes in the country, most pipes are imported from India. uPVC pipes conform to either NS 206 – 2046 or IS 4985 – 2000. They are available in pipe diameters ranging from 20 mm to over 300 mm and in pressure classes ranging from 2.5 – 15 kg/cm2. Use of such pipes in pipe network systems in Nepal is relatively new.

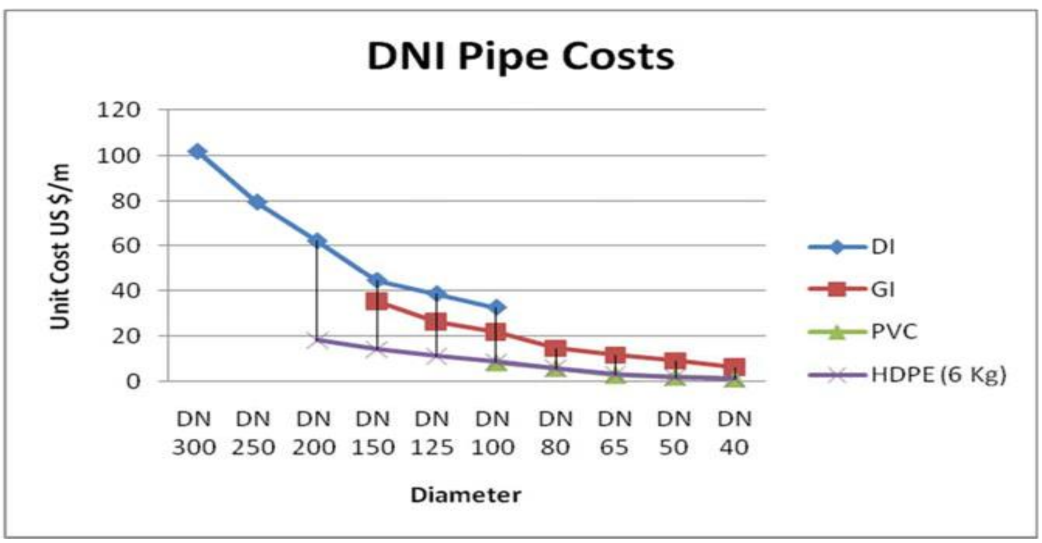
5 Comparative Assessment of Pipe Materials

The four pipe materials discussed have their own distinct advantages and disadvantages. However, for the present purposes the pipe materials have been comparatively assessed in terms of the following factors – transportation; laying and joining; strength, operation and maintenance (including availability of spare parts) and initial investment costs. The comparative table below summarizes the advantages and disadvantages of the four pipe materials.

Pipe Material	Advantages	Disadvantages
DI – ISO 2531	• Pipes are relatively strong and can withstand large external loads with modest bedding. • The majority of standard pipes and fittings can handle pressures up to 16 Bar. • The flexible joint system means that earthquake movements can be accommodated. • The strength of the pipes means they are less likely to be damaged. • Ductile iron pipes can be laid quicker with short lengths of trench causing less disruption to road traffic. • The system can be modified and additional facilities easily added to the pipe network. • Ductile iron pipes are semi-rigid pipes and generally do not require bedding to be of as high a standard as PE or PVC pipes.	• Restrained joint can be provided by most manufactures where the pipes are to withstand some longitudinal forces; however, generally anchor (thrust) blocks are required. • Unit weight of pipes is generally higher than PE and PVC Pipes. • Transportation cost can be relatively higher. • Per unit cost of DI pipes in comparison with GI Pipes is similar, but becomes cheaper for larger diameter pipes.
PE100 / HDPE	• PE100 pipes are relatively lighter in unit weight and easier to transport. • The internal surface is much smoother resulting in higher “C” value or lower resistance. • Repair and maintenance is relatively easy • More suited for conveyance of potable water. • Greater laying flexibility and requires average bedding requirements.	• The pressure rating of the larger diameter pipes is limited. • Without good storage conditions plastic pipes deteriorate, and storage of materials for future repairs may be difficult. • Butt-fusion welding of joints needs special equipment and skill. • Susceptible to damage by rodents and creatures of that nature.

GI Pipes	• Higher working pressure • Available in diameters ranging from 15mm to 150 mm • GI Pipes are readily available in Nepal and are locally manufactured. • Local plumbers are conversant in the use and installation of GI pipes. • Not susceptible to damage by rodents and other such creatures. • Generally resistant to external weather conditions	• Poorly galvanized pipes very susceptible to corrosion and hence leakage. • Leakage from badly joined pipe ends and fittings. • Relatively heavier than uPVC and PE pipes. • Susceptible to underground soil conditions and requires better protection.
uPVC Pipes	•uPVC pipes are relatively lighter in unit weight and easier to transport. • The internal surface is much smoother resulting in higher "C" value or lower resistance. • Suited for conveyance of potable water. •uPVC pipes are rigid enough for threading and joining works similar to GI pipes.	• The pressure rating of the larger diameter pipes is limited. • To ensure the pipeline has good structural properties, a good standard of bedding is required. • Without good storage conditions plastic pipes deteriorate and future storage of materials for repairs may be difficult. • Laying and joining of uPVC pipes requires better skills and joining materials.

A comparison of the three pipe materials in relation to their unit costs is presented in the chart below.



As it is evident from the chart, DI pipe is more cost effective for large diameter pipes of distribution systems. While the other pipes; PE, GI and uPVC, of diameters above 150mm need to be specially ordered. In terms of cost, DI and GI pipes are more costly, while PE and uPVC pipes are relatively cheaper.

6 Pipe Material Recommendation for Distribution Network

For the present DNI works it is recommended that all pipes of sizes DN 150 and above shall be DI. While for smaller diameter pipes including those for house connections, non- metallic pipes – i.e. PE and uPVC are recommended for use. As the quality of galvanization of locally available GI pipes usually is not of good standards, use of GI pipes should be avoided to the maximum extent possible.

Annex 2 - TERMS OF REFERENCE (TOR) FOR IEE

Objective: To guide the proponent or consultant in the preparation of Environmental Assessment report of the desired quality.

Why:

- Listing of activities to be performed
- Systematizing working procedures
- Specific activities to be performed
- Fitting the study within the policy and legal context
- Accomplishing the work within the time frame
- Providing technical guidance to the proponent/consultant

When:

- TOR approval by concerned authority before carrying out EA.

Content:

1. Introduction: background and purpose of the proposal, study boundary, responsible party for preparing the EA report, policy and legal requirements, and EA-related guidelines.
2. Scope of work to be considered during the study.
3. Alternatives
4. Institutional and public involvement: how they should be involved.
5. Required information: major tasks, study schedule, reviews, study team, costs, data and information.
6. Analysis of impacts: Positive and negative impacts, identification, prediction and evaluation of impacts using necessary methods and techniques.
7. Impact mitigation and monitoring: environmental management plan, monitoring plan and monitoring costs.
8. Conclusions and recommendations
9. References
10. Annexes
11. Need for Executive Summary (both in English and Nepali).

Department of Water Supply and Sewerage Management **Asian Development Bank**

Ministry of Water Supply
Government of Nepal

URBAN WATER SUPPLY AND SANITATION PROJECT (UWSSP)
GRANT NO. - NEP

INITIAL ENVIRONMENTAL EXAMINATION FOR **XXX** SUB-PROJECT

TERMS OF REFERENCE

CONTENTS

Initial Environmental Examination

Terms of Reference for the XXX Water Supply, Drainage and Sanitation Projects

1. Organization preparing the Terms of Reference

The Environment Protection Regulations of 1997 have been amended on 5 April 1999 wherein it has been mentioned that a ToR for an IEE is required.

This ToR for the Initial Environmental Examination has been prepared by YYY in for the Government of Nepal, Ministry of Water Supply, Department of Water Supply and Sewerage Management (DWSSM). This ToR is a part of the consulting services as agreed to between the proponent, DWSSM and the Consultant.

2. Description of the Project and Project Background

Describe the existing water supply and sanitation systems and drainage of the town.

3. Methodology

- Prepare a comprehensive database on the corridor of influence on the bio-physical and the socio-economic environment.
- Secondary data will have to be collected from published and unpublished reports, maps, aerial photographs, newspaper articles, etc. from different Governmental and non-governmental organizations.
- Questionnaires/checklists/matrices for collection of primary data will be prepared for both the bio-physical and socio-economic assessments.
- Consultant will provide a description of relevant parts of the Project, using maps with appropriate scale and photographs and aerial photos where necessary, including the following information: location, alignment and alternatives, design standards, pre- construction activities, construction activities, post-construction activities, work schedule, staffing and support facilities and services.
- Information on mitigation costs associated with construction activities (during design, construction and operation and maintenance activities) should also be included.

3.1 Environmental assessment

- Existing environmental constraints and potential impacts in the Project area have to be studied through field surveys, complemented by secondary information from reports and interviews with a number of government officials, representatives of NGO and International Organizations (IO) supported projects and researchers.
- The Consultant will collect primary and secondary data, evaluate them and describe the relevant environmental characteristics of the area along the

- pipeline routes and its corridor of influence, including the following information:

(a) **Physical Environment**: topography, soils, climate and meteorology, geology, surface and ground water hydrology, noise, air and water quality

(b) **Biological Environment**: flora, fauna, rare and endangered species, religious trees and sensitive habitats (including parks or reserves)

- The Consultant will develop all necessary documents for field visit and collect data with the help of the survey team. It is suggested that the IEE team go to the field and work as a team and not dispersed at different times.

3.2 Socio-economic assessment

- Social assessment of the projects tries to determine the social implications (issues) in terms of assumed positive and negative impacts related to location, design, construction and operation. Preparation and actual implementation of the construction activities will create some nuisance and inconvenience for the communities in the area.
- Primary data should be obtained through Focus Group Discussion (FGD) with communities along the pipeline routes under consideration. Additional data should be collected from the various Committees (VDCs, DDCs, NGOs, community groups etc.) through whose territory the respective pipe alignments pass.
- The Consultant will collect primary and secondary data, evaluate them and describe the relevant environmental characteristics of the area along the proposed alignment and its corridor of influence, including the following information:

  population, land use, planned development activities, community structure, government services, demography, employment, distribution of income and source of livelihood, goods and services produced, water supply, public health, education, extension services, cultural sites and heritage, tribal people, customs, aspirations and attitudes, expected water users and those benefiting from it, different needs and demands of VDCs, and the present Quality of Life (QoL) etc.

4. Policies, laws, rules and directives

- The Consultant will describe the pertinent regulations and standards that govern environmental quality, health and safety, protection of sensitive areas and endangered species etc. at international, regional, district, VDC and Ward levels.

- Nepal is a signatory to many international conventions, including those concerning habitat, bio-diversity and cultural heritage protection. These issues should be considered during the IEE and their avoidance/ mitigation measures should be identified.
- The IEE should also be conducted in compliance with the following GON Acts, Regulations and Guidelines:
- Policy Guidelines of Ninth Five Year Plan, HMG/N, Planning Commission, 1998
- 1998/1999 Fiscal Policy Guidelines, Ministry of Finance, HMG/N, 1998
- Environment Protection Act, 1996
- Environment Protection Regulations, 1997
- First Amendment to the Environment Protection Regulations, 1999
- National EIA Guidelines, 1993
- Draft EIA Guidelines for the Road Sector, 1997
- EIA Guidelines for the Forestry Sector, 1995
- Forest Act, 2049 and Forest Regulations, 2050
- Land Acquisition Act, 2034
- The IEE will also take into consideration ADB's "Safeguard Policy Statement" (SPS 2009).

5. Time, estimated budget, and specialists required

5.1 Time

After the approval of the ToR, it is expected that the approximate time needed to complete the final IEE report will be 6 weeks as follows:

	ACTIVITY	Duration
1.	Desk top study	1 week
2.	Field Study	2 weeks
3.	Data Compilation	1 week
4.	Draft IEE Report	1 week
5.	Final IEE Report	1 week

5.2 Estimated Budget

A budget of approximately Rs. ……..will be required to complete the IEE for the Project.

5.3 Specialists/Expertise required

The IEE studies require a multidisciplinary team of experts for the bio-physical and socio-economic assessments. The following team is proposed:

- Environmental Specialist
- Geologist
- Botanist/Forester

- Sociologist
- Water Supply and Sanitary Engineer

Three to four enumerators will also be required to help the team.

6. Scope of work

The Bank's Guidelines and MoPE's "Environment Protection Regulations 2054" broadly define the scope of work required in the IEE.

7. Anticipated impacts of the Proposed Project

A distinction will have to be made between potentially significant positive and adverse impacts, direct and indirect impacts and immediate and long-term impacts. Impacts that are unavoidable or irreversible will have to be identified. Wherever possible, the significant impacts are to be quantified in terms of environmental costs and benefits.

Potential physical, biological and social impacts should be considered.

8. Analysis of Alternatives to the Proposed Project

Alternative alignments to the proposed project to meet the same project objectives will have to be described (siting, design, technology choice, construction techniques, operation and maintenance). Alternatives in terms of potential environmental impacts, capital and operating costs and institutional training and monitoring requirements should be described. Costs and benefits of each alternative should be quantified (wherever possible), incorporating the estimated costs of any associated mitigation measures.

9. Mitigation Measures

Mitigation measures for adverse potential impacts due to location, design, construction and post-construction will have to be proposed. Mitigation measures will have to be incorporated from the planning stage onwards. These measures should be outlined in the Environmental Management Plan (EMP) and the Resettlement Plan (RP).

10. Development of an Environmental Management Plan to Mitigate Adverse Impacts

An Environmental Management Plan (EMP) has a dual purpose. It is designed to monitor the contractor's work during project implementation. It helps to check contractual compliance with specified mitigation measures. It also helps in making periodic checks on the actual environmental impacts of the Project over the years following completion of the works, and compare these with those impacts anticipated at the time of Project appraisal.

The EMP therefore provides the necessary feedback required for correcting potentially serious Project deficiencies, and for planning of other projects. Feasible and cost-effective measures to prevent/mitigate/reduce significant negative impacts should be recommended in an Environmental Monitoring Management Plan, outlining construction and post-construction measures. The impacts and costs associated with

implementing the measures will have to be detailed. Issues related to compensation of affected parties for impacts that cannot be mitigated will have to be considered. The EMP will include proposed work programs, budget estimates, schedules, staffing and training requirements and other support services to implement the mitigating measures.

A detailed Resettlement Action Plan (covering compensation and/or resettlement) for the Project Affected Persons should be prepared in addressing the socio-economic impacts.

11. Reports

The format for the IEE report should include the following:

1. Executive summary (in English and Nepali)
2. Table of contents
3. List of tables
4. List of figures/photographs
5. Appendices
6. List of Abbreviations
7. Acknowledgement
8. Introduction
9. Description of the project
10. Relevancy of the project
11. Description of the environment
12. Anticipated environmental impacts and mitigation measures
13. Analysis of alternatives with and without project situations
14. Information disclosure, consultation, and participation
15. Grievance and redress mechanism
16. Environmental management plan
17. Conclusions and recommendations
18. References
19. Annexes

12. Relevant Information

Tables, figures, maps, photographs and references that should be included in the IEE report.

Annex 3 - IEE REPORT FORMAT

Executive Summaries in Nepali and English

A. INTRODUCTION

1. Name and address of the individual institution preparing the report

a. Name of the proposal
b. Name and address of the proponent
c. Consultant preparing the report

2. Basis and extent of the IEE study

a. ADB Policy
b. National Laws, Policies, Acts, Regulations, Standards and Guidelines
c. Objectives and Scope of the Environmental Study
d. Relevancy of the Project
e. Approach and Methodology

DESCRIPTION OF THE PROJECT

3. Existing Water Supply, Sanitation and Drainage Infrastructure
    a. Water Supply
    b. Sanitation
    c. Drainage
4. Type, category and need of the Subprojects
5. Size or magnitude of operation
6. Proposed schedule for implementation
7. Description of the Subprojects

Table: Components of Subprojects

Infrastructure	Function	Description	Location

C. DESCRIPTION OF THE ENVIRONMENT

1. Physical Resources

Topography

Geology and soils,

seismology Climate and air

quality Water Resources

Surface water and

quality Groundwater

and quality

2. Ecological Resources

National Parks and protected areas

Forests (including rare or endangered species)

Flora

Fauna

Fisheries/aquatic biology

3. Social and cultural resources

Population and

communities Health

facilities Educational

facilities

Socio-economic conditions (community structure, family structure, social well-being)

Physical or cultural

heritage Employment

Slums and Squatter Settlements

4. Economic Development and Prospects for Growth

Land Use Infrastructure

Transportation

Drinking Water

Supply Surface

Drainage, Sanitation

& Sewerage

Electricity Communications

Economic Characteristics

Industries

Agricultural development

Mineral development

Tourism development

Development Organizations

Major Environmental

Problems Health and

Sanitation

D. ANTICIPATED ENVIRONMENTAL IMPACTS AND MITIGATION MEASURES

1. Pre-construction Phase

a. Environmental impacts due to project design

A. Construction Phase

a. environmental impacts due to project construction

i. Physical Environment

ii. Biological Environment

iii. Socio-Economic and Cultural Environment

Compensation and rehabilitation as per the Resettlement Plan (RP)
Reinstatement of damaged community services and infrastructure Influx of outside workforce, money and unwanted activities

Health and safety

a. Occupational Health and Safety (OHS) b . Community Health and Safety

c . Dislocation of archaeological artifacts

d. Traffic management

B. **Operational Phase**

Table: Summary of Mitigation Measures for Subproject Components

Project Stage	Project Activity	Potential Environmental Impacts	Proposed Mitigation Measures	Institutional Responsibility	Cost (Rs)
Pre-Project Activity (Project Design)					
Preparation for construction					
Construction Phase: Physical Environment					
Construction Phase: Biological Environment					
Construction Phase: Socio-Economic Environment					
Operational Phase					

E. Analysis of alternatives with and without project situations

F. Information disclosure, consultation, and participation

G. Grievance and redress mechanism

H. matters to be monitored while implementing the project

ENVIRONMENTAL MANAGEMENT PLAN (EMP)

Table 1: Monitoring Requirements

Impacts/Project Activities							
Environmental Impact	Mitigation Measures	Parameters to be Monitored	Location	Measurements	Responsibility	Frequency	Cost
Design Phase							
Pre-Construction Activities							
Construction Phase: Physical Environment							
Construction Phase: Biological Environment							
Construction Phase: Socio-economy							
Operation Phase							

Mitigation and monitoring

a. *Construction Phase*
b. *Operation Phase*

1. Environmental Procedures and Institutions
2. Grievance and Redress Mechanism
3. Potential Environmental Enhancement Measures
4. Reporting Procedures
5. Procurement Plan and Cost Estimates
6. Work Plan

I. CONCLUSIONS AND RECOMMENDATIONS

Annex 4- RAPID ENVIRONMENTAL ASSESSMENT (REA) CHECKLISTS FOR WATER SUPPLY

Instructions:

(i) The project team completes this checklist to support the environmental classification of a project. It is to be attached to the environmental categorization form and submitted to the Environment and Safeguards Division (RSES) for endorsement by the Director, RSES and for approval by the Chief Compliance Officer.

(ii) This checklist focuses on environmental issues and concerns. To ensure that social dimensions are adequately considered, refer also to ADB's (a) checklists on involuntary resettlement and Indigenous Peoples; (b) poverty reduction handbook; (c) staff guide to consultation and participation; and (d) gender checklists.

(iii) Answer the questions assuming the "without mitigation" case. The purpose is to identify potential impacts. Use the "remarks" section to discuss any anticipated mitigation measures.

Country/Project Title:	
Sector Division	

SCREENING QUESTION	Yes	No	Remarks
A. Project Siting Is the project area …			
▪ Densely populated?			
▪ Heavy with development activities?			
▪ Adjacent to or within any environmentally sensitive areas?			
• Cultural heritage site			
• Protected Area			
• Wetland			
• Mangrove			
• Estuarine			
• Buffer zone of protected area			
• Special area for protecting biodiversity			
• Bay			
B. Potential Environmental Impacts Will the project cause…			
• Pollution of raw water supply from upstream wastewater discharge from communities, industries, agriculture, and soil erosion runoff?			
• Impairment of historical/cultural monuments/areas and loss/damage to these sites?			
• Hazard of land subsidence caused by excessive ground water pumping?			
• Social conflicts arising from displacement of communities?			
• Conflicts in abstraction of raw water for water supply with other beneficial water uses for surface and ground waters?			
• Unsatisfactory raw water supply (e.g. excessive pathogens or mineral constituents)?			
• Delivery of unsafe water to distribution system?			
• Inadequate protection of intake works or wells, leading to pollution of water supply?			

• Over pumping of ground water, leading to salinization and ground subsidence?			
• Excessive algal growth in storage reservoir?			
• Increase in production of sewage beyond capabilities of community facilities?			
• Inadequate disposal of sludge from water treatment plants?			
• Inadequate buffer zone around pumping and treatment plants to alleviate noise and other possible nuisances and protect facilities?			
• Impairments associated with transmission lines and access roads?			
• Health hazards arising from inadequate design of facilities for receiving, storing, and handling of chlorine and other hazardous chemicals			
• Health and safety hazards to workers from handling and management of chlorine used for disinfection, other contaminants, and biological and physical hazards during project construction and operation?			
• Dislocation or involuntary resettlement of people?			
• Disproportionate impacts on the poor, women and children, Indigenous Peoples or other vulnerable groups?			
• Noise and dust from construction activities?			
• Increased road traffic due to interference of construction activities?			
• Continuing soil erosion/silt runoff from construction operations?			
• Delivery of unsafe water due to poor O&M treatment processes (especially mud accumulations in filters) and inadequate chlorination due to lack of adequate monitoring of chlorine residuals in distribution systems?			
• Delivery of water to distribution system, which is corrosive due to inadequate attention to feeding of corrective chemicals?			
• Accidental leakage of chlorine gas?			
• Excessive abstraction of water affecting downstream water users?			
• Competing uses of water?			
• increased sewage flow due to increased water supply			
• increased volume of sullage (wastewater from cooking and washing) and sludge from wastewater treatment plant			
• Large population influx during project construction and operation that causes increased burden on social infrastructure and services (such as water supply and sanitation systems)?			
• Social conflicts if workers from other regions or countries are hired?			
• Risks to community health and safety due to the transport, storage, and use and/or disposal of materials such as explosives, fuel and other chemicals during operation and construction?			
• community safety risks due to both accidental and natural hazards, especially where the structural elements or components of the project are accessible to members of the affected community or where their failure could result in injury to the community throughout project construction, operation and decommissioning			

Climate Change and Disaster Risk Questions **The following questions are not for environmental categorization. They are included in this checklist to help identify potential climate and disaster risks.**	**Yes**	**No**	**Remarks**
• Is the Project area subject to hazards such as earthquakes, floods, landslides, tropical cyclone winds, storm surges, tsunami or volcanic eruptions and climate changes (see Appendix I)?			
• Could changes in temperature, precipitation, or extreme events patterns over the Project lifespan affect technical or financial sustainability (e.g., changes in rainfall patterns disrupt reliability of water supply; sea level rise creates salinity intrusion into proposed water supply source)?			
• Are there any demographic or socio-economic aspects of the Project area that are already vulnerable (e.g., high incidence of marginalized populations, rural-urban migrants, illegal settlements, ethnic minorities, women or children)?			
• Could the Project potentially increase the climate or disaster vulnerability of the surrounding area (e.g., by using water from a vulnerable source that is relied upon by many user groups, or encouraging settlement in earthquake zones)?			

* Hazards are potentially damaging physical events.

Annex 5 - RAPID ENVIRONMENTAL ASSESSMENT (REA) CHECKLISTS FOR SEWAGE TREATMENT

Instructions:

(i) The project team completes this checklist to support the environmental classification of a project. It is to be attached to the environmental categorization form and submitted to the Environment and Safeguards Division (RSES) for endorsement by the Director, RSES and for approval by the Chief Compliance Officer.

(ii) This checklist focuses on environmental issues and concerns. To ensure that social dimensions are adequately considered, refer also to ADB's (a) checklists on involuntary resettlement and Indigenous Peoples; (b) poverty reduction handbook;
(c) staff guide to consultation and participation; and (d) gender checklists.

(iii) Answer the questions assuming "without mitigation" case. The purpose is to the identify potential impacts. Use "remarks" section to discuss any anticipated the mitigation measures.

Country/Project Title:

Sector Division

Sewage Treatment

Screening Questions	Yes	No	Remarks
A. Project Siting Is the Project area ….			
▪ Densely Populated?			
▪ Heavy with development activities?			
▪ Adjacent to or within any environmentally sensitive areas?			
• Cultural heritage site			
• Protected Area			
• Wetland			
• Mangrove			
• Estuarine			
• Buffer Zone			
• Special area for protecting biodiversity			
B. Potential Environmental Impacts Will the Project cause…			
▪ Impairment of historical/cultural monuments /area and loss/damage to these sites?			
▪ Interference with other utilities and blocking of access to buildings; nuisance to neighboring areas due to noise, smell and influx of insects, rodents, etc.?			
▪ Dislocation or involuntary resettlement of people?			
▪ Disproportionate impacts on the poor, women and children, indigenous peoples or other vulnerable groups?			
▪ Impairment of downstream water quality due to inadequate sewage treatment or release of untreated sewage?			
▪ Overflows and flooding of neighboring properties with raw sewage?			
▪ Environmental pollution due to inadequate sludge disposal or industrial waste discharges illegally disposed in sewers?			
▪ Noise and vibration due to blasting and other civil works?			

▪ Risks and vulnerabilities related to occupational health and safety due to physical, chemical, and biological hazards during project construction and operation?			
▪ Discharge of hazardous materials into sewers, resulting in damage to sewer system and danger to workers?			
▪ Inadequate buffer zone around pumping and treatment plants to alleviate noise and other possible nuisances, and protect facilities?			
▪ Road blocking and temporary flooding due to land excavation during the rainy season?			
▪ Noise and dust from construction activities?			
▪ Traffic disturbances due to construction material transport and wastes?			
▪ Temporary silt runoff due to construction?			
▪ Hazards to public health due to overflow flooding, and groundwater pollution due to failure of sewerage system?			
▪ Deterioration of water quality due to inadequate sludge disposal or direct discharge of untreated sewage water?			
▪ Contamination of surface and ground waters due to sludge disposal on land?			
▪ Health and safety hazards to workers from toxic gases and hazardous materials which may be contained in confined areas, sewage flow and exposure to pathogens in untreated sewage and un-stabilized sludge?			
▪ Large population increase during project construction and operation that causes increased burden on social infrastructure (such as sanitation system)?			
▪ Social conflicts between construction workers from other areas and community workers?			
▪ Risks to community health and safety due to the transport, storage, and use and/or disposal of materials such as explosives, fuel and other chemicals during construction and operation?			
▪ Community safety risks due to both accidental and natural hazards, especially where the structural elements or components of the project are accessible to members of the affected community or where their failure could result in injury to the community throughout project construction, operation and decommissioning?			

Sewage Treatment

Climate Change and Disaster Risk Questions The following questions are not for environmental categorization. They are included in this checklist to help identify potential climate and disaster risks	**Yes**	**No**	**Remarks**
▪ Is the Project area subject to hazards such as earthquakes, floods, landslides, tropical cyclone winds, storm surges, tsunami or volcanic eruptions and climate changes (see Appendix I)?			
▪ Could changes in precipitation, temperature, salinity, or extreme events over the Project lifespan affect its sustainability or cost?			
▪ Are there any demographic or socio-economic aspects of the Project area that are already vulnerable (e.g. high incidence of marginalized populations, rural-urban migrants, illegal settlements, ethnic minorities, women or children)?			
▪ Could the Project potentially increase the climate or disaster vulnerability of the surrounding area (e.g., increasing traffic or housing in areas that will be more prone to flooding, by encouraging settlement in earthquake zones)?			

Appendix I: Environments, Hazards and Climate Changes

Environment	**Natural Hazards and Climate Change**
Arid/Semi-arid and desert environments	Low erratic rainfall of up to 500 mm rainfall per annum with periodic droughts and high rainfall variability. Low vegetative cover. Resilient ecosystems & complex pastoral and systems, but medium certainty that 10–20% of drylands degraded; 10-30% projected decrease in water availability in next 40 years; projected increase in drought duration and severity under climate change. Increased mobilization of sand dunes and other soils as vegetation cover declines; likely overall decrease in agricultural productivity, with rain- fed agriculture yield reduced by 30% or more by 2020. Earthquakes and other geophysical hazards may also occur in these environments.
Humid and sub-humid plains, foothills and hill country	More than 500 mm precipitation/yr. Resilient ecosystems & complex human pastoral and cropping systems. 10-30% projected decrease in water availability in next 40 years; projected increase in droughts, heatwaves and floods; increased erosion of loess-mantled landscapes by wind and water; increased gully erosion; landslides likely on steeper slopes. Likely overall decrease in agricultural productivity & compromised food production from variability, with rain-fed agriculture yield reduced by 30% or more by 2020. Increased incidence of forest and agriculture-based insect infestations. Earthquakes and other geophysical hazards may also occur in these environments.

River valleys/ deltas and estuaries and other low-lying coastal areas	River basins, deltas and estuaries in low-lying areas are vulnerable to riverine floods, storm surges associated with tropical cyclones/typhoons and sea level rise; natural (and human-induced) subsidence resulting from sediment compaction and ground water extraction; liquefaction of soft sediments as result of earthquake ground shaking. Tsunami possible/likely on some coasts. Lowland agri-business and subsistence farming in these regions at significant risk.
Small islands	Small islands generally have land areas of less than 10,000km^2 in area, though Papua New Guinea and Timor with much larger land areas are commonly included in lists of small island developing states. Low- lying islands are especially vulnerable to storm surge, tsunami and sea-level rise and, frequently, coastal erosion, with coral reefs threatened by ocean warming in some areas. Sea level rise is likely to threaten the limited ground water resources. High islands often experience high rainfall intensities, frequent landslides and tectonic environments in which landslides and earthquakes are not uncommon with (occasional) volcanic eruptions. Small islands may have low adaptive capacity and high adaptation costs relative to GDP
Mountain ecosystems	Accelerated glacial melting, rockfalls /landslides and glacial lake outburst floods, leading to increased debris flows, river bank erosion and floods and more extensive outwash plains and, possibly, more frequent wind erosion in intermontane valleys. Enhanced snow melt and fluctuating stream flows may produce seasonal floods and droughts. Melting of permafrost in some environments. Faunal and floral species migration. Earthquakes, landslides and other geophysical hazards may also occur in these environments.
Volcanic environments	Recently active volcanoes (erupted in last 10,000 years – see www.volcano.si.edu). Often fertile soils with intensive agriculture and landslides on steep slopes. Subject to earthquakes and volcanic eruptions including pyroclastic flows and mudflows/lahars and/or gas emissions and occasionally widespread ashfall.

Annex 6- RAPID ENVIRONMENTAL ASSESSMENT (REA) CHECKLISTS FOR SOLID WASTE MANAGEMENT

Instructions:

- ❑ This checklist is to be prepared to support the environmental classification of a project. It is to be attached to the environmental categorization form that is to be prepared and submitted to the Chief Compliance Officer of the Regional and Sustainable Development Department.

- ❑ This checklist is to be completed with the assistance of an Environment Specialist in a Regional Department.

- ❑ This checklist focuses on environmental issues and concerns. To ensure that social dimensions are adequately considered, refer also to ADB checklists and handbooks on
  (i) involuntary resettlement, (ii) indigenous peoples planning, (iii) poverty reduction,
  (iv) participation, and (v) gender and development.

- ❑ Answer the questions assuming the "without mitigation" case. The purpose is to identify potential impacts. Use the "remarks" section to discuss any anticipated mitigation measures.

Country/Project Title:	
Sector Division	

Solid Waste Management

Screening Questions	Yes	No	Remarks
A. Project Siting Is the Project area ….			
▪ Densely Populated?			
▪ Heavy with development activities?			
▪ Adjacent to or within any environmentally sensitive areas?			
• Cultural heritage site			
• Protected Area			
• Wetland			
• Mangrove			
• Estuarine			
• Buffer Zone			
• Special area for protecting biodiversity			
• Bay			
B. Potential Environmental Impacts Will the Project cause…			
▪ Impacts associated with transport of wastes to the disposal site or treatment facility			
▪ Impairment of historical/cultural monuments/areas and loss/damage to these sites?			
▪ Degradation of aesthetic and property value loss?			
▪ Nuisance to neighboring areas due to foul odor and influx of insects, rodents, etc.?			
▪ Dislocation or involuntary resettlement of people			
▪ Public health hazards from odor, smoke from fire, and diseases transmitted by flies, insects, birds and rats?			
▪ Deterioration of water quality as a result of contamination of receiving waters by leachates from land disposal system?			
▪ Contamination of ground and/or surface water by leach ate from land disposal system?			
▪ Land use conflicts?			
▪ Pollution of surface and ground water from leach ate coming from sanitary landfill sites or methane gas produced from			

Solid Waste Management

decomposition of solid wastes in the absence of air, which could enter the aquifer or escape through soil fissures at places far from the landfill site?			
▪ Inadequate buffer zone around landfill site to alleviate nuisances?			
▪ Social conflicts between construction workers from other areas and community workers?			
▪ Road blocking and/or increased traffic during construction of facilities?			
▪ Noise and dust from construction activities?			
▪ Temporary silt runoff due to construction?			
▪ Hazards to public health due to inadequate management of landfill site caused by inadequate institutional and financial capabilities for the management of the landfill operation?			
▪ Emission of potentially toxic volatile organics from land disposal site?			
▪ Surface and ground water pollution from leach ate and methane gas migration?			
▪ Loss of deep-rooted vegetation (e.g. tress) from landfill gas?			
▪ Explosion of toxic response from accumulated landfill gas in buildings?			
▪ Contamination of air quality from incineration?			
▪ Public health hazards from odor, smoke from fire, and diseases transmitted by flies, rodents, insects and birds, etc.?			
▪ Health and safety hazards to workers from toxic gases and hazardous materials in the site?			